GEOTECHNICAL INVESTIGATION
Stevens Creek Levee Recertification
Mountain View, California

Submitted to:
Schaaf & Wheeler, Santa Clara, California

Submitted by:
AMEC Geomatrix, Inc., Oakland, California

May 2009

Project 13815.002

AMEC Geomatrix

May 1, 2009

Project 13815.002

Mr. Chuck Anderson
Schaaf & Wheeler, Consulting Civil Engineers
100 N. Winchester Blvd. Suite 200
Santa Clara, CA 95050

Subject: **Geotechnical Investigation**
Stevens Creek Levee Recertification
Mountain View, California

Dear Mr. Anderson:

AMEC Geomatrix, Inc. (AMEC) is pleased to submit this Geotechnical Investigation Report to support the Stevens Creek Levee Recertification project in Mountain View, California. This report was developed in accordance with our Agreement for Professional Services with Schaaf & Wheeler, dated April 15, 2008 (fully executed May 22, 2008). Our investigation included compiling and reviewing existing data, performing a field exploration, performing geotechnical laboratory testing, performing engineering evaluations and analyses, and preparing this report.

We conclude that we are not aware of any geotechnical considerations that we consider to stand in the way of recertification of the levee in accordance with 44 CFR 65.10.

If you have any questions about this report, please do not hesitate to call any of the undersigned. It has been a pleasure working with you and we look forward to working with you on other future phases of the project.

Sincerely yours,
AMEC Geomatrix, Inc.

Jian Hu, PE
Project Engineer

James B. French, PE, GE
Senior Engineer

REGISTERED PROFESSIONAL ENGINEER
JAMES B. FRENCH
No. 2018
Exp. 6/30/10
GEOTECHNICAL
STATE OF CALIFORNIA

David Ritzman, PE, GE
Senior Engineer

Faiz I. Makdisi, PE
Vice President and Principal Engineer

JH/JBF/DR/FIM/LU
\\Oad-fs1\doc_safe\13000s\13815.002\3000 Report\FINAL Stevens Creek Levees Rpt\1 txt, cvrs\Stevens Creek Cvrltr.doc

Enclosure

AMEC Geomatrix, Inc.
2101 Webster Street, 12th Floor
Oakland, CA
USA 94612-3066
Tel (510) 663-4100
Fax (510) 663-4141
www.amecgeomatrixinc.com

AMEC Geomatrix

TABLE OF CONTENTS

Page

1.0 INTRODUCTION 1
1.1 PURPOSE AND SCOPE 1
1.2 PROJECT TEAM 2
1.3 REPORT ORGANIZATION 2
2.0 LEVEE DESCRIPTION 2
2.1 PAST LEVEE PERFORMANCE 4
3.0 SITE INVESTIGATION 5
3.1 DATA REVIEW 5
3.2 FIELD RECONNAISSANCE 6
3.3 FIELD EXPLORATION 7
3.3.1 Exploratory Borings 7
3.3.2 Cone Penetration Tests (CPTs) 7
3.4 GEOTECHNICAL LABORATORY TESTING 8
4.0 GEOLOGIC AND SEISMIC SETTING 8
4.1 REGIONAL GEOLOGY 8
4.2 SEISMICITY 9
5.0 AERIAL PHOTOGRAPH REVIEW 11
5.1.1 2/24/1956 Photographs 11
5.1.2 12/5/1960 Photographs 12
5.1.3 11/11/1969 Photographs 12
5.1.4 12/24/1975 Photographs 12
5.1.5 1/28/1977 Photographs 12
5.1.6 2/28/1980 Photographs 13
5.1.7 10/28/1980 Photographs 13
5.1.8 3/5/1981 Photographs 13
5.1.9 6/10/1981 Photographs 13
5.1.10 12/16/1992 Photographs 13
5.1.11 9/21/1994 Photographs 14
5.1.12 2/28/2008 Photographs 14
6.0 LEVEE FILL AND FOUNDATION CONDITIONS 14
6.1 SITE GEOLOGIC CONDITIONS 14
6.2 SUBSURFACE CONDITIONS 14
6.2.1 Levee Fill 15
6.2.2 Upper Clay 15
6.2.3 Sand 15
6.2.4 Lower Clay 15
6.3 GROUNDWATER CONDITIONS 15
7.0 GEOTECHNICAL ANALYSES 16
7.1 LIQUEFACTION ASSESSMENT 16
7.1.1 Analysis Methodology 17
7.1.2 Results of the Liquefaction Analysis 18
7.2 SELECTION OF CROSS SECTIONS FOR ANALYSES 18
7.2.1 Cross Section Geometry 18

TABLE OF CONTENTS
(Continued)

Page

7.2.2 Cross Section Stratigraphy 19
7.2.3 Material Properties for Design 19
7.3 SEEPAGE ANALYSIS 20
7.3.1 Initial Conditions 21
7.3.2 Boundary Conditions 21
7.3.3 Hydraulic Conductivity 22
7.3.4 Steady State Seepage Analysis Results 22
7.3.5 Transient Seepage Analysis 23
7.4 SLOPE STABILITY ANALYSIS 23
7.4.1 Existing Condition 24
7.4.2 Steady State Seepage Condition 25
7.4.3 Rapid Drawdown Condition 25
7.4.4 Seismic Stability Analysis 26
7.4.5 Flood Wall Stability Evaluation 27
7.5 SETTLEMENT EVALUATION 29
7.5.1 Static Settlement 29
7.5.2 Seismic Settlement 29
8.0 CONCLUSIONS 29
8.1 EROSION OF LEVEE EMBANKMENT DURING BASE FLOOD 29
8.2 LEVEE EMBANKMENT STABILITY 29
8.2.1 Seismic Stability and Deformation 30
8.3 LEVEE FOUNDATION STABILITY 30
8.4 ENCROACHMENTS/MAINTENANCE 30
9.0 LIMITATIONS 31
10.0 REFERENCES 32

TABLES

Table 1 Material Properties
Table 2 Summary of Analysis Results

TABLE OF CONTENTS
(Continued)

FIGURES

Figure 1 Site Location Map
Figure 2 Levee Alignments and Field Exploration Map, Southern Reach
Figure 3 Levee Alignments and Field Exploration Map, Central Reach
Figure 4 Levee Alignments and Field Exploration Map, Northern Reach
Figure 5 Regional Fault Map
Figure 6 East Levee Profile
Figure 7 West Levee Profile

Figure 8-1 Reach A - Steady-State Seepage Analysis
Figure 8-2 Reach B - Steady-State Seepage Analysis
Figure 8-3 Reach C - Steady-State Seepage Analysis
Figure 8-4 Reach D - Steady-State Seepage Analysis

Figure 9-1 Reach B - Transient Seepage Analysis, Time = 0 hr
Figure 9-2 Reach B - Transient Seepage Analysis, Time = 14.4 hr
Figure 9-3 Reach B - Transient Seepage Analysis Time = 24 hr

Figure 10-1 Reach A - Existing Slope Stability Analysis, Waterside
Figure 10-2 Reach B - Existing Slope Stability Analysis, Landside
Figure 10-3 Reach B - Existing Slope Stability Analysis, Waterside
Figure 10-4 Reach C - Existing Slope Stability Analysis, Landside
Figure 10-5 Reach C - Existing Slope Stability Analysis, Waterside
Figure 10-6 Reach D - Existing Slope Stability Analysis, Landside
Figure 10-7 Reach D - Existing Slope Stability Analysis, Waterside

Figure 11-1 Reach B - Steady-State Slope Stability Analysis, Landside
Figure 11-2 Reach C - Steady-State Slope Stability Analysis, Landside
Figure 11-3 Reach D - Steady-State Slope Stability Analysis, Landside

Figure 12-1 Reach A - Drawdown Slope Stability Analysis, Waterside
Figure 12-2 Reach B - Drawdown Slope Stability Analysis, Waterside
Figure 12-3 Reach C - Drawdown Slope Stability Analysis, Waterside
Figure 12-4 Reach D - Drawdown Slope Stability Analysis, Waterside

Figure 13-1 Reach A - Seismic Screening Analysis (0.15g), Waterside
Figure 13-2 Reach B - Seismic Screening Analysis (0.15g), Landside
Figure 13-3 Reach B - Seismic Screening Analysis (0.15g), Waterside
Figure 13-4 Reach C - Seismic Screening Analysis (0.15g), Landside
Figure 13-5 Reach C - Seismic Screening Analysis (0.15g), Waterside
Figure 13-6 Reach D - Seismic Screening Analysis (0.15g), Landside
Figure 13-7 Reach D - Seismic Screening Analysis (0.15g), Waterside

Figure 14-1 Reach A - Seismic Yield Determination, Waterside
Figure 14-2 Reach B - Seismic Yield Determination, Landside
Figure 14-3 Reach B - Seismic Yield Determination, Waterside
Figure 14-4 Reach C - Seismic Yield Determination, Landside
Figure 14-5 Reach C - Seismic Yield Determination, Waterside
Figure 14-6 Reach D - Seismic Yield Determination, Landside
Figure 14-7 Reach D - Seismic Yield Determination, Waterside

TABLE OF CONTENTS
(Continued)

APPENDICES

Appendix A Previous Data
Appendix B Field Reconnaissance Memorandum
Appendix C Field Exploration
Appendix D Laboratory Test Results
Appendix E Summary of Liquefaction Analysis Results
Appendix F Stevens Creek Design Flood Stage and Hydrographs

GEOTECHNICAL INVESTIGATION REPORT
Stevens Creek Levees
FEMA Levee Certification
Mountain View, California

1.0 INTRODUCTION

AMEC Geomatrix, Inc. (AMEC, formerly known as Geomatrix Consultants, Inc. [Geomatrix]) is pleased to present the results of our geotechnical investigation for recertification of the Stevens Creek east and west bank levees between Highway 101 and Crittenden Lane in the City of Mountain View, California. The location of the project is shown on the attached Site Location Map, Figure 1.

1.1 PURPOSE AND SCOPE

This investigation was performed on behalf of the Santa Clara Valley Water District (the District) and the City of Mountain View; our work was performed under subcontract to Schaaf & Wheeler, Consulting Civil Engineers (S&W) to evaluate the stability of the levees under existing, steady-state flood, rapid drawdown, and seismic loading, and to contain the base flood from a geotechnical standpoint. This study is part of a broader effort to evaluate the levees using the criteria set forth in 44 CFR Section 65.10 of the National Flood Insurance Program (NFIP), which includes minimum certification standards for levee design, operations, and maintenance. These standards must be maintained for the levees to remain accredited by FEMA as providing protection from the estimated 100-year flood. This report strictly addresses the geotechnical aspects of the FEMA levee certification process; other engineering evaluations, such as hydrologic and hydraulic analyses to satisfy the FEMA criteria are not addressed in this report.

The purpose of our work has been to provide geotechnical data, analyses and documents to support a professional opinion about whether the existing levees on both banks of Stevens Creek from Crittenden Lane to Highway 101 in Mountain View meet the requirements of 44 CFR 65.10.

The scope of services to accomplish the above-stated purpose was outlined in the Professional Services Agreement (PSA) between AMEC and S&W, dated April 15, 2008 (fully executed May 22, 2008). Tasks performed by AMEC in support of this purpose have included the following:

1) Compiled geotechnical data to describe surface and subsurface conditions, including the engineering properties of levee fill and foundation materials.

2) Performed geotechnical engineering analyses to evaluate liquefaction, seepage, embankment stability, settlement, and geotechnical aspects of the stability of the flood wall. Specific analyses have included the following:

   a) Perform geotechnical analyses to evaluate liquefaction and levee stability per USACOE EM 1110-2-1913.

   b) Complete a levee seepage analysis.

   c) Complete a stability evaluation.

   d) Complete a settlement evaluation.

3) Prepared this Report to document the findings of the geotechnical field exploration, and analyses.

1.2 PROJECT TEAM

The project was directed by S&W as the prime consultant. Under subcontract to S&W, AMEC work included compiling and reviewing available pertinent geotechnical data, performing field reconnaissance, performing geotechnical engineering analyses, and preparing this report. Geotechnical borings and cone penetration tests (CPTs), as well as geotechnical laboratory tests were performed by Pacific Geotechnical Engineers (PGE), also under subcontract to S&W; their work was coordinated with S&W and AMEC. PGE's work will be presented under separate cover, but the boring logs, CPT logs, and laboratory test results are presented as appendices to this report.

1.3 REPORT ORGANIZATION

The project description, scope of work, and project team are described in Section 1.0. An overview of the levees is presented in Section 2.0. The methods of investigation are described in Section 3.0. The site conditions, including the regional geologic and seismic settings, are presented in Section 4.0. The history of the levee, based on a review of aerial photographs, is presented in Section 5.0. Surface and subsurface conditions, site geologic setting, groundwater conditions, and seismic site classification are described in Section 6.0. A discussion of our geotechnical analyses is presented in Section 7.0, and our conclusions are presented in Section 8.0. General limitations and references are presented in Sections 9.0 and 10.0, respectively.

Appendices present background data as well as supplemental documentation of our analyses.

2.0 LEVEE DESCRIPTION

From its origins in the Santa Cruz Mountains west of San Jose, California, Stevens Creek flows northeast approximately 20 miles to the southern end of San Francisco Bay. Upper reaches of the creek flow in a southeasterly direction through the mountains to Stevens Creek

Reservoir before turning to the north and entering the Santa Clara Valley. Lower reaches of the creek flow northward through the cities of Cupertino, Sunnyvale, and Mountain View before discharging to the bay. North of Highway 101, the Stevens Creek channel is bounded by levees on the east and west sides of the creek.

As shown on Figure 1, the study area for this project follows Stevens Creek for approximately 5,900 feet (1.1 miles) between Highway 101 to the south and Crittenden Lane to the north. The study area includes levees on the east and west sides of the creek, herein termed the east bank levee and west bank levee, respectively. Immediately upstream of the study area, Stevens Creek flows through the heavily developed, urban environment of the northern Santa Clara Valley. Downstream of the study area, the creek continues for approximately 1.5 miles past existing salt ponds, tidal marshes, and wetlands before reaching the open waters of San Francisco Bay. Tidal influence from the bay extends into the northern portion of the study area.

Further north within the study area, the east and west bank levees generally consist of earthen embankments, except for an approximately 1,600-foot-long stretch on the east bank where the levee is replaced by a concrete flood wall. Development on the east side of the creek includes former military housing associated with the Moffett Federal Airfield and the NASA Ames Research Center. Development on the west side of the creek includes commercial offices, a trailer park, and a plant nursery.

Levees along the east and west banks of Stevens Creek within the study area were originally constructed prior to the 1956 air photos. In the early 1980s, the levees were relocated, raised, and widened by the District as part of a comprehensive grading and levee improvement project. In addition to constructing new levees, that improvement project also included dredging and shaping the creek channel, constructing the flood wall, and raising the Crittenden Lane Bridge by approximately 10 feet to create additional flow capacity within the channel (Berlogar, 1981). Pertinent portions of this report and other previous data are reproduced in Appendix A.

Aerial photographs of the levees, along with the District's stationing line, are shown on Figures 2 through 4. Locations of features described in this report are referenced to the District's levee stationing system; however, the references to District stationing presented in this report are approximate. Stevens Creek enters the study area at the point where it discharges from a rectangular box culvert beneath Highway 101, near Station 110+40. The creek exits the study area at the point it flows beneath the Crittenden Lane Bridge, near Station 169+50.

After the creek passes north through the channel beneath Highway 101, the east bank levee extends from a short distance north of the north side of Highway 101 to the right abutment of the Crittenden Lane Bridge. The crest of the levee is gravel-surfaced and serves as an access road for levee maintenance and is a pedestrian/bicycle trail. Between Stations 126+80 and 142+50, the east bank levee is replaced by a concrete flood wall that ranges from approximately 1 to 8 feet in height above the land side ground surface elevations. In this area, the gravel-surfaced access road is located adjacent to the wall on the waterside. A detailed description and photographs of the east bank levee are provided in Appendix B.

The west bank levee extends from a short distance north of the north side of Highway 101 to the left abutment of the Crittenden Lane Bridge. With the exception of a relatively short section located immediately north of Highway 101, the crest of the levee is paved with asphalt-concrete and serves as an access road for levee maintenance and is a pedestrian/bicycle trail. A detailed description of the west bank levee is provided in Appendix B.

Based on available data of cross sections and observations made during our field reconnaissance, existing slopes at the site appear to have maximum heights of approximately 15 feet on the waterside and 13 feet on the landside of the levee, with typical inclinations of 2:1 (horizontal: vertical) and crest widths of 10 ft.

We observed evidence of animal burrowing (apparently mostly gophers and possibly some ground squirrels) in a number of locations, as documented in Appendix B. We recommend regularly scheduled maintenance of the subject levees to fill these holes and to remove the animals making them. Additional observations are presented in Appendix B.

A detailed review of levee pipe penetrations has been made by S&W in conjunction with this study. A summary of the major penetration is provided in Appendix B.

2.1 PAST LEVEE PERFORMANCE

It is our understanding that the project levees have performed quite well since construction, with no major erosion or sliding problems. It is our understanding that this reach of Stevens Creek has survived several significant floods with no major damage. A "regional" analysis of the 20 largest floods on record for San Francisquito Creek at nearby Stanford University reveals the following (note that the last major storm was 12/31/2005, with a ~15-year return interval):

Ranking	Date	Peak Discharge (cfs)	Approx. Return Period (years)
1	2/3/98	7,200	80
2	12/22/55	5,560	30
3	1/04/82	5,220	20
4	12/31/05	4,840	15
5	4/02/58	4,460	10
6	1/21/67	4,000	8
7	2/13/00	3.930	7
8	2/16/82	3,760	7
9	12/16/02	3,660	7
10	11/18/50	3,650	7
11	2/17/86	3,480	6
12	1/26/83	3,420	6
13	4/01/74	3,410	6
14	1/16/73	3,390	6
15	1/24/83	3,340	5
16	1/09/95	3,320	5
17	1/13/80	3,300	5
18	1/31/63	3,270	5
19	1/02/97	3,250	5
20	1/13/83	3,240	5

Flood levels in 1998 might have been only a foot or two lower than design levels at the most. Also, there was a very high coincident tide on that day, so water levels may have been close to design levels in the lower reaches. There were no recorded performance problems in 1998.

3.0 SITE INVESTIGATION

The site investigation performed for the project consisted of the following tasks:

- Data Review
- Field Reconnaissance
- Field Exploration (performed by PGE)
- Geotechnical Laboratory Testing (performed by PGE)

A summary of the site investigation methods are described in more detail below. PGE will provide a report under separate cover documenting the field and laboratory investigation.

3.1 DATA REVIEW

AMEC compiled and reviewed readily available geologic and geotechnical investigation reports concerning the subject levees and contained in AMEC's project files and in the files of the Santa Clara Valley Water District. The primary information obtained during the data review include previous borings by Porter, Urqhart, McCreary, and O'Brien Consultants (Porter, 1958)

shown on a plan and profile sheet for the Stevens Creek project and a previous geotechnical investigation report prepared by Berlogar, Long and Associates (Berlogar, 1981) for the previous Stevens Creek flood control improvements. Approximate exploration locations identified in previous reports are shown on Figures 2 through 4. Logs of these previous exploration locations and the results of relevant laboratory tests are provided in Appendix A.

In additional to the existing geologic/geotechnical investigation reports, AMEC reviewed published maps and reports to evaluate geologic and geotechnical data relevant to the project site. These sources consisted of readily available geologic and fault maps prepared by the United States Geological Survey (USGS, 2006a, 2006b) and the California Geological Survey (CGS, 2002).

AMEC also reviewed 12 sets of historical black-and-white aerial photographs with a stereoscope to observe past land use and broad geomorphologic features at and around the subject levees for information pertinent to the geologic/geotechnical conditions. The photographs were taken in 1956, 1960, 1969, 1975, 1977, 1980, 1981, 1992, 1994, and 2008 and had scales ranging from 1:1,200 to 1:12,000. Aerial photographs supplied by and were reviewed at the offices of the Santa Clara Water District. Summaries of the conditions observed on the photographs are provided in Section 5.0.

Detailed citations of these published and unpublished sources are compiled in the references section at the end of this report.

3.2 FIELD RECONNAISSANCE

On May 28 through May 30, 2008, and on June 2, 2008, AMEC performed field reconnaissance of the subject levees to observe overall site and levee conditions, including levee geometry, and whether any obvious geologic/geotechnical concerns were exposed.

General reconnaissance activities included the following:

- Visually observing the levees, creek channel, and adjacent areas for evidence of settlement, erosion, ground deformation/cracking, and potential stability problems;
- Approximately measuring levee dimensions and slope inclinations;
- Identifying the presence and condition of erosion protection measures, outfalls, closures, floodwalls, and other structures/improvements along the levee alignments;
- Identifying trees, utilities, and other features that encroach on levee slopes;
- Observing water levels and flow conditions in the creek channel;

- Photographing the levees, adjacent areas, and other features of interest; and
- Locating utilities, penetrations, and other features of interest using handheld Global Positioning System (GPS) equipment.

Conditions observed during the field reconnaissance were documented in levee field reconnaissance logs, which also include brief descriptions of photographs and GPS locations collected during the reconnaissance effort. Findings of the field reconnaissance, including field logs, photographs, and GPS locations and feature descriptions, were presented in a draft memorandum submitted to S&W, dated August 15, 2008. The final site reconnaissance memorandum is included as Appendix C to this report.

3.3 FIELD EXPLORATION

A field exploration program, consisting of drilling exploratory borings and performing cone penetration tests (CPTs), was performed by PGE to explore the subsurface conditions on the subject levees. The field exploration program is described in more detail in the following sections.

3.3.1 Exploratory Borings

Four borings (Borings B-1 through B-4) were drilled on the subject levees on September 15 and September 16, 2008. The exploratory borings were advanced to depths of approximately 30 to 70 feet below the top of the levees at the approximate locations shown on Figures 2 through 4.

The borings were performed under the direction of PGE using either hollow-stem auger or mud-rotary wash drilling techniques. Subsurface materials were sampled using conventional modified-California (mod-Cal) and Standard Penetration (SPT) split-spoon samplers driven with a 140-pound hammer dropping 30 inches in general conformance with ASTM D1586 procedures and conventional thin-walled Shelby tube samplers that were hydraulically pushed into the ground. The field exploration program is described in more detail in Appendix C. Logs of subsurface conditions encountered in the borings, prepared by PGE, are also presented in Appendix C.

3.3.2 Cone Penetration Tests (CPTs)

Twenty CPT probes were used to explore the subsurface conditions at the site between September 8 and September 10, 2008. The CPT probes were advanced by John Sarmiento and Associates, of Orinda, California, working under subcontract to PGE, to depths of approximately 23 to 70 feet below the top of the levees. The approximate locations of the CPTs are shown on Figures 2 through 4. Graphic logs are presented in Appendix C.

3.4 GEOTECHNICAL LABORATORY TESTING

Laboratory tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. Samples were tested for dry density, moisture content, Atterberg limits, grain size distribution, undrained shear strength, and consolidation. The laboratory test program was prepared by PGE. The laboratory tests were performed by PGE and by Cooper Testing Labs of Palo Alto, California, under subcontract to PGE). Tabular and graphic presentations of the test results are presented in Appendix D.

4.0 GEOLOGIC AND SEISMIC SETTING

4.1 REGIONAL GEOLOGY

The San Francisco Bay Area is located at the boundary between the Pacific and North American plates, two large crustal plates that are separated by the north-northwest-trending San Andreas Fault, within the California Coast Ranges Geomorphic Province. The geomorphology of the region includes parts of three prominent, northwest-trending geologic/geomorphic features, which include from west to east the Santa Cruz Mountains, Santa Clara Valley, and the Diablo Range. Santa Clara Valley forms part of an elongated structural block (the San Francisco Bay block) within the central Coast Ranges that contains San Francisco Bay and its surrounding alluvial margins (Page, 1989). This structural block is fault bounded by the San Andreas Fault to the southwest and the Hayward-Calaveras fault zone to the northeast (see Figure 5).

The oldest rocks in the region belong to the Franciscan Complex of Jurassic to Cretaceous age (205 to 65 million years before present [Ma]). These rocks are intensely deformed (i.e., folded, faulted, and fractured) due to ancient tectonic processes and, to a lesser extent, from more recent tectonic processes associated with the San Andreas Fault system. Franciscan rocks generally comprise the “basement” of the Coast Ranges northeast of the San Andreas Fault; Cretaceous granitic rocks, known as the Salinian block, comprise the basement of the ranges located southwest of the San Andreas Fault. A sequence of Tertiary (65 to 1.8 Ma) marine and nonmarine sedimentary rocks unconformably overlies, and locally is in fault contact with the granitic and Franciscan basement rocks in the region.

Quaternary (1.6 Ma to present) surficial deposits are concentrated in the Santa Clara Valley and locally overlie the complexly deformed Cretaceous rocks and Tertiary strata in the adjacent hills. During the Plio-Pleistocene (5 Ma to 11,000 years ago [ka]) epochs, sediments eroded from the uplifting Diablo Range and the Santa Cruz Mountains formed broad alluvial fan complexes along the margins of Santa Clara Valley. The 5 Ma to 300,000-year-old (Plio-Pleistocene) Santa Clara Formation, which consists of a sequence of fluvial and lucustrine sediments, was deposited unconformably on the older Tertiary and Franciscan rocks along the

margins of Santa Clara Valley during this time and subsequently has been folded, faulted, and eroded. Although the deposition of the Santa Clara Formation spans a relatively long interval and the stratigraphy of the formation is complex, it has been used as a “marker horizon” for evaluating the recency of faulting in the Santa Clara Valley region. The Santa Clara Formation is unconformably overlain by younger Quaternary and Holocene (11 ka to present) alluvial and fluvial deposits (stream channel, overbank, and flood basin environments), which interfinger to the north with estuarine muds of San Francisco Bay (Helley et al., 1979).

South San Francisco Bay is a north-northwest-trending subsiding basin that is filled primarily with Quaternary alluvium (stream) deposits eroded from the surrounding margins and estuarine (bay mud). The Sangamon and Holocene bay muds are separated by the Quaternary alluvium and eolian (wind blown) sand deposits. Alluvium deposits consist of sediments eroded from the surrounding Santa Cruz Mountains and Diablo Range uplands. These alluvial sediments were transported and deposited by streams and include a mixture of sands, gravels, silts, and clays with highly variable permeability. In contrast, the fine-grained bay muds have very low permeability. The youngest Holocene bay muds underlie almost all of the original San Francisco Bay (Atwater et al., 1977; Helley et al., 1979), including the subject levees.

Due to movement on the San Andreas and related faults including the Hayward and Calaveras Faults, as well as the previous geologic history, a wide variety of igneous, metamorphic, and sedimentary rocks are present.

Geologic units are classified according to geologic time periods. The most recent time period is the Quaternary Period, which covers from 1.8 million years ago to present. The Quaternary Period is subdivided into the Pleistocene Epoch (1.8 Ma to 11 ka) and the Holocene Epoch (11 ka to present). The Pleistocene includes a number of worldwide glacial periods of low sea level stands. Estuarine (bay) muds have been deposited in San Francisco Bay during high sea level periods of the Sangamon (70 ka to 130 ka) and the Holocene (less than 11 ka) (Atwater et al., 1977).

4.2 SEISMICITY

San Francisco Bay is considered to be one of the more seismically active regions in the world, based on its record of historic earthquakes and its position astride the San Andreas Fault system (see Figure 5). The San Andreas Fault system consists of several major right-lateral strike-slip faults in the region that define the boundary zone between the Pacific and North American tectonic plates. During the past 200 years, faults within this plate boundary zone have produced numerous small-magnitude and at least fifteen moderate to large (i.e., M > 6) earthquakes affecting the Bay Area (Toppozada et al., 1981; Ellsworth, 1990; Bakun, 1999).

The U.S. Geological Survey (USGS) 2007 Working Group on California Earthquake Probabilities (WGCEP, 2008) estimated an approximately 63 percent probability that at least one major moment magnitude earthquake ($M_W \geq 6.7$) would occur in the San Francisco Bay Area before 2037, with a 21 percent probability that that such an earthquake will occur on the San Andreas Fault.

A large earthquake occurred in June 1838 on the Peninsula segment of the San Andreas Fault (Ellsworth, 1990). Toppozada and Borchardt (1998), Hall et al. (1999) and Hall et al. (2001), and Bakun (1999) reevaluated the data for this earthquake and estimated (local) magnitudes of 7.5, 7.0 to 7.4, and 6.8, respectively. Severe shaking of Modified Mercalli Intensity (MMI) VIII to IX occurred in the vicinity of San Jose during this event, with houses shaken down and damage to walls of adobe structures (Toppozada and Borchardt, 1998). The MMI is an experience-based rating system used to categorize the intensity of shaking (prior to use of the seismograph), and is based on the relative damage done to structures/property and general uneasiness of the population as a result of the earthquake.

An earthquake having an estimated M_L of 6.1 (Bakun, 1999) occurred in the San Jose region on November 26, 1858. This earthquake resulted in cracking of almost every brick, adobe, or concrete building in San Jose (Townley and Allen, 1939), corresponding to MMI VII to VIII (Toppozada et al., 1981).

A strong earthquake occurred on October 8, 1865, and apparently was centered in the Santa Cruz Mountains south of San Jose. This earthquake is estimated to have been of M_L 6.5 (Bakun, 1999), and caused damage to buildings in San Francisco, Santa Clara, San Jose, and other areas (Townley and Allen, 1939). The reported damage from the 1865 earthquake corresponds to MMI VII to IX in the Santa Clara, New Almaden, and San Jose areas (Toppozada et al., 1981).

The 1868 Hayward earthquake occurred north of San Jose along the southern segment of the Hayward Fault and had an estimated magnitude of 7.0 (WGCEP, 2008). Surface rupture apparently extended from Oakland southward to the Warm Springs area of Fremont. The event reportedly resulted in damage and/or the complete destruction of every building in Hayward, as well as damage to buildings as far south as Gilroy (Townley and Allen, 1939). The reported damage in Santa Clara County corresponds to MMI VII to VIII.

During the M_W 7.8 1906 San Francisco earthquake, the San Andreas Fault ruptured from Shelter Cove near Cape Mendocino southward to near San Juan Bautista. Maximum lateral displacements of 15 to 20 feet occurred north of the Golden Gate at Olema in Marin County (Lawson, 1908). Landslides, liquefaction, and ground settlement occurred throughout the

San Francisco Bay region and in the vicinity of the surface rupture as result of this earthquake. Extensive damage occurred to many buildings in San Jose, corresponding to MMI VIII to IX (Toppozada and Parke, 1982). Youd and Hoose (1978) also report that ground failures resulting from the 1906 earthquake were locally observed in the coarse gravelly bottom of Coyote Creek.

More recent earthquakes in the region include the 1957 Daly City earthquake on the San Andreas Fault (M_L 5.3); the Coyote Lake and Morgan Hill earthquakes of 1979 and 1984 on the Calaveras Fault (M_L 5.9 and 6.1, respectively); the 1980 Livermore earthquake on the Greenville Fault (M_L 5.8); and the 1989 M_L 7.1 Loma Prieta earthquake on the San Andreas Fault or a parallel subsidiary fault. Of these earthquakes, the strongest shaking and most damage resulted from the October 17, 1989, M_L 7.1 Loma Prieta earthquake.

The 1989 earthquake ruptured on or southwest of the Santa Cruz Mountains segment of the San Andreas Fault and produced MMI VII effects in San Jose. No liquefaction or associated ground-failure effects related to the Loma Prieta earthquake were reported in the vicinity of the subject levees by Tinsley et al. (1998). The closest liquefaction or associated ground-failure effects related to the Loma Prieta earthquake were reported along Guadalupe River in Alviso approximately 5 miles (8 km) east of the subject levees, where settlement occurred in the approach fills of the Gold Street bridge; no earthquake-related cracks were observed in the dikes (levees) in the northwest part of Alviso by Tinsley et al. (1998). The Seismic Hazards Zones map of the Mountain View Quadrangle (CGS, 2003) shows the subject levees within the zone of potential liquefaction. A Liquefaction Susceptibility map covering the subject levees indicates that the subject levees are underlain by Holocene fine-grained alluvial fan and basin deposits (Qhff) with a moderate liquefaction susceptibility (Witter, et al., 2006, Sheet2).

5.0 AERIAL PHOTOGRAPH REVIEW

A summary of the levee history, as interpreted from our aerial photograph review, is presented below. It should be noted that each summary only presents significant changes from each prior photograph date.

5.1.1 2/24/1956 Photographs

This set of photographs at a scale of ≈ 1 inch =100 feet provides incomplete stereo coverage. The creek is essentially in its present location. Low rounded, possibly natural levees are present downstream of the Crittenden Lane bridge (present). Upstream of the bridge, levees appear to be present, and a general cross section from east to west consists of levee, channel, levee, and toe berm. Except for some trees at the bend in the creek between about

Stations 125+00 and 127+00, the channel and levees are non-vegetated. Some dumped fill may locally be present on the creek banks.

5.1.2 12/5/1960 Photographs

This set of photographs at a scale of ≈ 1 inch = 100 feet provides incomplete stereo coverage. The creek appears to be more channelized down stream of the Crittenden Lane bridge and levees may be present. Upstream of the bridge, the general cross section appears to be similar to 1956, and conditions are similar except for the construction of creek bank protection downstream of the Highway 101 overcrossing, and the construction of concrete drop structures in the channel at about Stations 111+50, 119+00 and 127+00.

5.1.3 11/11/1969 Photographs

This set of photographs at a scale of ≈ 1 inch = 500 feet provides stereo coverage between about Stations 110+00 and 138+00. Essentially no trees or vegetation are present. Bank erosion appears to be locally present, particularly downstream of the concrete bank protection downstream of the Highway 101 overcrossing. A low degraded levee appears to be present on the east side of the creek between about Stations 118+00 and 122+00 adjacent to housing. Concrete bank protection is present downstream of the concrete drop structure at about Station 119+00. The creek appears to be wider downstream of about Station 128+00. The concrete drop structure at about Station 127+00 appears to have been removed between this date and 1977.

5.1.4 12/24/1975 Photographs

This set of photographs at a scale of ≈1 inch = 250 feet provides incomplete stereo coverage between about Station 110+00 and 170+00. The creek appears to be a narrow channel within a marsh or low flood terrace. A degraded (possibly older) levee is present on the east side of the creek between about Stations 148+00 and 151+00. A small swale (or possible slump) is present at the top of the creek bank at about Station 123+00.

5.1.5 1/28/1977 Photographs

This set of photographs at scales of ≈ 1 inch = 500 feet and ≈1 inch = 1000 feet provides incomplete stereo coverage. On the less detailed photographs, furrows, probably dredged channel spoil, are present on the creek banks perpendicular to the channel between about Stations 140+00 and 151+00. On the more detailed photographs, similar furrows are present on the creek banks between about Stations 119+00 and 127+00. The channel in these areas extends bank to bank, consistent with channel dredging. The dredging may be related to levee construction/repair.

5.1.6 2/28/1980 Photographs

This set of photographs at a scale of ≈ 1 inch = 250 feet provides incomplete stereo coverage. The creek appears to be in flood upstream of the Crittenden Lane bridge. The channel side of the levee on the east side of the creek upstream of about Station 122+00 appears to be eroded. The levee on the east side of the creek upstream of the Crittenden Lane bridge to about Station 143+00 is visible, but upstream of about Station 143+00 a levee, if present, is very low and this area may have a narrow flood wall. Upstream of about Station 150+00 the furrows on the creek banks are overgrown.

5.1.7 10/28/1980 Photographs

This set of photographs at a scale of ≈1 inch = 250 feet provides coverage between about Stations 122+00 and 143+00. The creek bank on the east side of the creek is overgrown. Extensive grading is present on the creek bank on the west side of the creek and in the channel between about Stations 129+00 and 135+00 and a berm is being constructed along the toe of the creek bank on the east side of the creek

5.1.8 3/5/1981 Photographs

This set of photographs at a scale of ≈ 1 inch = 250 feet provides coverage between about Stations 122+00 and 143+00. The berm constructed along the toe of the creek bank on the east side of the creek diverts the channel to the creek bank along this interval.

5.1.9 6/10/1981 Photographs

This set of photographs at a scale of ≈ 1 inch – 250 feet provides incomplete stereo coverage. The creek bank and creek side levee slope on the west side of the creek upstream of the Crittenden Lane bridge looks ragged and etched. The crest of this levee appears to be locally below the top of the levee shoulders. A non-vegetated berm appears to be present along the toe of the creek bank on the west side of the creek. On the east side of the channel, vegetation is present on the channel/berm bank and on the creek bank.

5.1.10 12/16/1992 Photographs

This set of photographs at a scale of ≈ 1 inch = 500 feet provided incomplete stereo coverage. Conditions are similar to present conditions. Upstream of the Crittenden Lane bridge, a general cross section from east to west consists of levee, berm with road, marsh or low flood terrace, channel, berm with road, levee, and berm with road. Drainage or other structures that may penetrate the levee are present on the east side of the channel at about Stations 111+50, 125+50,129+00, 136+50, and 159+50, and on the west side of the channel at about Stations 111+00, 136+50, 142+00, 155+00, and 159+50, A ramp connecting the top of the creek bank and bermon the east side of the creek descends downstream at about Station 125+00.

5.1.11 9/21/1994 Photographs

This set of photographs at a scale of ≈ 1 inch = 300 feet provides complete stereo coverage. Downstream of the Crittenden Lane bridge, a general cross section from east to west consists of levee, berm with road, marsh or low flood terrace, channel, vegetated berm, and levee. A riprap repair appears to be present on the channel/berm bank on the east side of the creek opposite the drainage structure at about Station 159+50. The flood wall between about Stations 126+00 and 142+00 is distinct. Concrete protection is present on the creek bank on the west side of the creek below the end of La Avenida Street. A levee is present on the east side of the creek between about Stations 120+00 and the Highway 101 overcrossing.

5.1.12 2/28/2008 Photographs

This set of photographs at a scale of ≈ 1 inch = 250 feet provides coverage between about Stations 110+00 to 127+00. Conditions are similar to present conditions.

6.0 LEVEE FILL AND FOUNDATION CONDITIONS

6.1 SITE GEOLOGIC CONDITIONS

A recent geologic map covering the subject levees by Dibblee (2007) indicates that the reach of Stevens Creek downstream of Highway 101 to about the salt evaporators and head of the Stevens Creek slough is mapped as underlain by alluvial deposits described as fine-grained sand, silt, and clay (Qya) representing distal alluvial fan deposits at the outer edge of fan areas. Downstream of the subject levees, Stevens Creek is mapped as underlain by estuarine deposits described as organic clay and silty clay (Qbm) representing San Francisco Bay Mud, and including areas modified to salt pond evaporation basins. The ground west and east of Stevens Creek is mapped as underlain by alluvial deposits described as fossiliferous silty clay and organic clay (Qac) representing intra-fan deposits.

A recent surficial deposits map covering the subject levees (Witter et al., 2006, Sheet 1) indicates that the subject levees are underlain by Holocene Bay Mud (Qhbm) downstream of about the Crittenden Lane bridge and fine-grained alluvial fan deposits (Qhff) upstream of the bridge.

6.2 SUBSURFACE CONDITIONS

The laboratory tests and subsurface exploration performed defined properties and sub-layers used in the analysis of the levee stability, seepage, and settlement. The subsurface material properties used for evaluation are tabulated in Table 1 with discussion in Section 7.2.3. The geotechnical materials encountered in the explorations consisted of Levee Fill over clayey alluvium over sand over a lower clay layer. Each material is described below.

6.2.1 Levee Fill

The fill layer thickness is approximately 8 to 15 feet depending on the reach cross section location. The fill layer generally consists of very stiff lean clay (CL) and fat clay (CH), with lesser amounts of medium dense to very dense clayey or silty sand (SC, SM) with some gravelly zones. (One sample in a previous boring, P-6, was identified as a poorly graded sand [SP], but the high moisture content means that it must have been fairly clayey rather than poorly graded.) The fill soil appears to have been derived from channel deposits dredged up onto the levee.

6.2.2 Upper Clay

A fine-grained alluvium clay layer underlies the fill layer, typically with an approximate thickness of approximately 25 to 30 feet. The layer consists of generally stiff to very stiff (and occasionally medium stiff) lean clay (CL), fat clay (CH), and scattered thin clayey sand zones. This layer consists of alluvial deposits. Consolidation tests in this material indicate that the layer is over consolidated.

6.2.3 Sand

Underlying the upper clay layer is a loose to dense sandy layer approximately 2 to 7 feet thick depending on the reach cross section. This alluvial layer consists of clayey sand (SC), well graded sand with gravel and clay (SW-SC), and silty sand (SM).

6.2.4 Lower Clay

The lower clay layer underlies the sand layer and generally consists of stiff to very stiff fat clay (CH) and lean clay (CL) with lenses of sandier material. This layer extended to the maximum depth explored in all borings and CPTs, and as such the thickness of the layer is unknown.

6.3 GROUNDWATER CONDITIONS

Groundwater was encountered at a depth of approximately 19 feet below the ground surface (bgs) in Boring B-1, approximately 6½ feet bgs in Boring B-3, and approximately 8 feet bgs in Boring B-4. Because mud-rotary methods were used to advance Boring B-2 the depth to groundwater could not be observed during drilling. Upon completion, all of the borings were backfilled to the ground surface with cement grout immediately after drilling. It should be noted that groundwater observations in the hollow-stem auger borings may have been made before the passage of sufficient time to allow groundwater conditions to come to equilibrium. In addition, fluctuations in the groundwater level may occur due to variations in rainfall, temperature, and other factors not evident at the time the observations were made.

Groundwater levels in previous investigations ranged from about 4 to17 feet bgs (Berlogar, 1981; Porter 1958).

7.0 GEOTECHNICAL ANALYSES

The subject levees were evaluated from a geotechnical standpoint in general conformance with the guidelines presented in 44 CFR 65.10. The following analyses were performed on critical cross sections thought to represent the worst case scenario along the levee alignment:

1) The potential for liquefaction triggering was assessed for the embankment fill and underlying alluvial soil deposits during a 475-year return period earthquake event. These analyses were performed assuming typical groundwater conditions (i.e., groundwater not elevated by flooding) due to the low probability of the 475-year earthquake event occurring concurrently with periods of high flooding.

2) Seepage conditions through and under the levee embankments were evaluated using the finite element computer software SEEP/W (GeoStudio, 2007). Due to the short term duration of the 100-year flood events, transient analyses to evaluate the seepage conditions within the levee and alluvial materials were performed.

3) The stability of the levee slopes was evaluated under both static and earthquake loading conditions using the finite element computer software Slope/W (GeoStudio, 2007). The static stability of the levee slopes was evaluated assuming flood loading conditions, and the seismic stability of the levee was evaluated assuming typical groundwater conditions (i.e., groundwater not elevated by flooding). Due to the low probability of the 475-year earthquake event occurring concurrently with periods of flooding, the impact of concurrent seismic and flood conditions was not evaluated; this is consistent with the Corps of Engineers Manual EM 1110-2-1913.

4) Settlement analysis included the evaluation of both long term static settlement and seismically-induced settlement (e.g., seismic compression).

In the southern or upstream reaches of the study area, south of about Station 125+00 on the east bank, and south of about Station 131+50 on the west bank, the adjacent ground beyond the stream channel was found to be higher than the flood elevations provided by S&W, as shown on Figures 6 and 7. Although there is an embankment along each bank, these are not considered to be flood protection levees under NFIP guidelines; therefore, no seepage or stability analyses were performed in these areas for our study.

7.1 LIQUEFACTION ASSESSMENT

Liquefaction is a soil behavior phenomenon in which a soil located below the groundwater surface loses a substantial amount of strength due to high excess pore-water pressure generated and accumulated during strong earthquake ground shaking. During earthquake ground shaking, induced cyclic shear creates a tendency in most soils to change volume by rearrangement of the soil-particle structure. The potential for excess pore-water pressure

generation and strength loss associated with this volume change tendency is highly dependent on the density of the soil, with greater potential in looser soils. Recently deposited (i.e., geologically young) and relatively loose natural soils, and uncompacted or poorly compacted artificial fills located below the groundwater table, are potentially susceptible to liquefaction. Loose sands and silty sands are particularly susceptible. Silty and clayey soils tend to be less susceptible than sandy soils to liquefaction-type behaviors. Even within sandy soils, the presence of finer-grained materials affects susceptibility. Dense natural soils and well-compacted fills have low susceptibility to liquefaction. Clayey soils and bedrock generally are generally not susceptible to liquefaction.

Permanent ground displacements due to lateral spreads or flow slides and differential settlement are commonly considered significant potential hazards associated with liquefaction. These displacement hazards are direct products of the soil behavior phenomena (i.e., high pore water pressure and significant strength reduction) produced by the liquefaction process. Lateral spreads are ground failure phenomena that occur near abrupt topographic features (i.e., free-faces such as creek banks) and on gently sloping ground underlain by liquefied soil. Lateral spreading movements may be on the order of inches to several feet or more and are typically accompanied by surface fissures and slumping.

Earthquake-induced settlement is a result of the dissipation of excess pore pressure generated by ground shaking that, as described above, is associated with the tendency for loose, saturated soils to rearrange into a denser configuration during shaking. Such dissipation will produce volume decreases (termed consolidation or compaction) within the soils that are manifested at the ground surface as settlement.

Saturated granular materials, potentially susceptible to liquefaction from strong earthquake shaking, were encountered sporadically beneath the project levees.

7.1.1 Analysis Methodology

Ground motions were estimated from the California Geological Survey (CGS) Probabilistic Hazard Assessment Model, 2002 (revised April 2003). The site latitude and longitude were used to find peak ground accelerations (PGA) a 475-year return period.

The deaggregated contribution to the probabilistic seismic hazard was estimated from the U.S. Geological Survey (USGS), 2002 Interactive deaggregations.

Liquefaction susceptibility was assessed using the software program CLiq v1.3 (GeoLogismiki, 2008). CLiq takes measured field CPT data and assesses liquefaction potential, and post-earthquake vertical settlement given a user-defined earthquake magnitude and PGA.

CLiq utilizes the NCEER method (Youd, et al., 2001) along with calibrated procedures for post-earthquake settlements and lateral displacements by Zhang et al. (2002 and 2004).

7.1.2 Results of the Liquefaction Analysis

The PGA for an alluvial site in the vicinity of Stevens Creek was estimated to be 0.48g, for a 475-year return period (CGS, 2003).

An earthquake magnitude of 7.9, corresponding to the modal magnitude of the deaggregated probabilistic motions, was used for liquefaction analysis (USGS, 2002).

Results of the liquefaction analyses indicate that the sandy soil layers encountered would likely liquefy during an earthquake event with a 475-year return period. A maximum permanent settlement of about 1.2 inches (CPT-07) was estimated to result from liquefaction during an earthquake event with a 475-year return period. It may be noted that many of the explorations conducted for this study encountered thinner zones of potentially liquefiable sandy soil material. Some explorations conducted for this study did not encounter any zones of potentially liquefiable soils. Liquefaction analyses output are included in Appendix E.

7.2 SELECTION OF CROSS SECTIONS FOR ANALYSES

7.2.1 Cross Section Geometry

Based on a consideration of geometry and subsurface conditions, four cross sections were selected for analyses, at locations shown on Figures 2 through 4 in plan view, and Figures 6 and 7 in profile. To represent the various levee reaches of the project, two sections were selected on the east levee and two on the west levee, with one on each levee being toward the northern portion of the project and two being further to the south.

Based on our review of the design flood elevations provided by S&W, and the elevations of the adjacent ground beyond the limits of the embankments, the adjacent ground is higher than the flood elevations south of Station 125+00 on the east bank and south of Station 131+50 on the west bank. We consider that the there is no levee present in these reaches and we did not perform any seepage or stability analyses in theses reaches.

We reviewed the as-built drawings and topographic cross sections provided by S&W at approximately 100-foot intervals along the project alignment. Within each reach, a cross section with the tallest and/or steepest slope was selected; this was considered generally to represent the most conservative geometry with respect to slope stability.

7.2.2 Cross Section Stratigraphy

Our interpreted stratigraphy along the centerline of each bank is shown on Figures 6 and 7 for the east bank and west bank, respectively. The stratigraphy of each cross section was developed to be representative of its reach rather than a strict interpretation at the location of the cross section.

The subsurface conditions encountered were relatively consistent along the length of levee each levee reach. As described in Section 6.2, the subsurface soils encountered during the subsurface exploration program generally consisted of embankment fill (or the flood wall along a portion of the east bank), underlain by alluvial soil deposits. Most of the alluvium consisted of clayey soils, including clays of varying plasticity and clayey sand. Within these clayey zones, a sand zone was present in most locations; the top of the sand layer was varied from about 25 to 30 below the original grade, and it was about 2 to 7 feet thick. The configuration of this sand layer was modeled in the cross sections to be representative of conditions in each reach.

7.2.3 Material Properties for Design

The primary material properties needed for analysis included unit weights, permeability, and strength. Properties for design are summarized on Table 1. These were selected as follows:

1) Unit weights for clayey zone were selected based on average values from field and laboratory data and consideration of published typical values. Unit weights for the sandy zone was selected based on a comparison published typical values for the type of material and calculated values based on laboratory water contents and an assumption of saturated conditions.

2) Permeabilities for clayey materials were selected based on calculations from the consolidation tests, published typical values for similar materials, and engineering judgment. Permeabilities for sandy material was selected based on calculations using the Kozeny-Carmen equations using the laboratory grain size distributions and published typical values for similar materials, and engineering judgment.

3) Strengths for the embankment material were selected based on a comparison of laboratory strength tests, field pocket penetrometer tests, CPT results, and typical correlations with SPT values. In general, values selected for analyses were quite conservative, nearly enveloping the lower bound of most of the data.

4) Strengths for the native clayey soils were also selected based on a comparison of laboratory strength tests, field pocket penetrometer tests, CPT results, and typical correlations with SPT values. In general, values selected for analyses were quite conservative, nearly enveloping the lower bound of most of the data.

5) Strengths for the native sandy soils were selected based on a comparison of laboratory results, and typical correlations with CPT and SPT values. In general, values selected for analyses were quite conservative.

The locations of the cross sections used for analyses are shown in Figures 2 through 4 as well as Figures 6 and 7. The cross sections are oriented looking downstream. In general, the crest width was about 10 feet, the embankment heights are typically about 14 feet for the waterside of the levee and 10 feet for the landside of the levee, and the maximum slope inclinations are about 2:1 (horizontal to vertical).

7.3 SEEPAGE ANALYSIS

Water, being driven by a large head differential, moving through or under an embankment causes increased seepage uplift forces in the embankment and/or the embankment foundation. If the weight of the embankment and/or foundation materials is not sufficient to restrain the seepage uplift forces acting on them, the materials have the potential for piping (erosion), which could lead to a loss of material and/or strength, thus impacting levee performance. It is therefore necessary to evaluate the effect that seepage during flood conditions may have on the on the landside of levees.

Under normal flow conditions, water in Stevens Creek flows in a pair of channels below the toe of the levees. During flood conditions, water can rise briefly against the waterside of the levees; during a 100-year flood the waterside slope is expected to experience water as high as about 4 feet below the levee crest. Two representative 100-year flood hydrographs are presented in Appendix F. As can be seen in these hydrographs, the duration of high water is only a matter of a few hours. Based on the short duration of the flood stage, it is unlikely that seepage will extend more than a few feet into the levee; however, as is a common and conservative practice with levee evaluations, we preformed stability calculations based on the assumption of steady state seepage, which yields conservative stability results that bound the worst case scenario; actual stability will be better than calculated.

In order to demonstrate the conservative nature of the steady state assumption, we performed a representative transient analysis of Reach B, as described below in Section 7.3.5. This analysis confirmed that the assumption of steady state seepage is quite conservative within the short duration of the design hydrograph.

As a simple and conservative evaluation of the potential for internal erosion or piping to occur, it is common to check that the he maximum average exit gradient at or downstream of the landside toe is less than 0.5 in a steady-state condition. The results of this check are discussed below in Section 7.4.2.

The computer program SEEP/W (GeoSlope, Ltd., 2007) is a two-dimensional, finite-element seepage analysis program. It was used to estimate the steady-state pore pressures within and underlying the levee, as well as to check for transient seepage conditions. The program

calculated steady state the gradients discussed below, and the resulting pore pressures were imported into the slope stability analysis cross sections. Finite-element representations of the surface and subsurface conditions for seepage analyses of the levee cross sections for Reach A through Reach D are provided on Figures 8-1 through 8-4, respectively.

Seepage analyses were performed for the four cross sections selected for analyses. The methodologies for determining the initial conditions and hydraulic conductivity for each analysis are explained below, as is the transient condition for Reach B.

7.3.1 Initial Conditions

The initial groundwater conditions were estimated from historical boring data and data from the subsurface exploration program. The groundwater elevations vary from 5 feet (toward the south end of the project) to 25 feet (toward the north) (NAVD 88), based on available water levels from the previous and current subsurface investigations. For our analyses, the groundwater elevation was assumed to be 4 feet above the invert of the channel throughout this section of Stevens Creek.

The flood water level conditions were based on hydrographs provided by S&W; these hydrographs are presented in Appendix F. Maximum flood stages were also provided by S&W and reproduced in Appendix F; these values are also shown on Figures 6 and 7. For our analyses, the flood level water level was simplified to be 4 feet below the levee crest, which is within about 1 foot of the elevations provided by S&W.

7.3.2 Boundary Conditions

Saturated and unsaturated zone permeabilities are calculated by the SEEP/W program using conductivity functions. For this analysis, general conductivity functions were chosen based on type of soils encountered in the subsurface exploration. Typical conductivity functions were then adjusted by SEEP/W to match the input saturated-flow permeabilities.

The following boundary conditions were applied to the model:

- On the waterside levee slope and channel bottom, a fixed total head boundary condition, corresponding to the analyzed water surface elevation, was applied along the waterside slope and channel.
- On the waterside model boundary, a vertical no flow boundary was applied.
- On the levee crest and landside slope, potential seepage face boundary (nodes that allow water to flow out of the model boundary) was applied and extended to the ground surface on the landside of the levee for the drawing extents. This allows SEEP/W to estimate a phreatic surface through the levee.

- On the bottom of the model, a no-flow boundary was applied.

Results of the steady state seepage analysis results are shown in graphical in Figures 8-1 through 8-4, and Table 2 shows the maximum gradient from the seepage analyses.

7.3.3 Hydraulic Conductivity

During the subsurface exploration program, soil samples of the levee fill and alluvial soils were retrieved; grain size distribution and consolidation tests were performed on select samples.

From the grain size distribution of the sandy material, the vertical permeability, k_v, was estimated using the Kozeny-Carman equations (Carrier, 2003). From the higher bound average of all sand grain size distributions for this layer the estimated saturated hydraulic conductivity is 1.2×10^{-2} cm/s. This value is on the high end of the calculation, hence conservative considering the variability in the soil types encountered during the field exploration program. Due to depositional anisotropy, we assumed the horizontal permeability, k_h, of 4.8×10^{-1} cm/s (i.e., $k_h / k_v \sim 4$).

With consolidation test results, permeability was calculated using the coefficient of consolidation c_v using the logarithm of time procedure by Casagrande and the square-root of time procedure by Taylor. Permeabilities were calculated using both procedures and the results were compared. This calculation suggests an average saturated vertical hydraulic conductivity, k_v, on the order of 2.8×10^{-7} cm/s for the clay layers. We assumed a ratio of $k_h / k_v \sim 10$ for the alluvial clays (assuming there will likely be siltier or sandier strata that will strongly influence horizontal permeability).

The fill layer soil type was assumed to range from very fine sand to a mixture of sand, silt, and clay with stratified clay deposits. From Holtz and Kovacs, a vertical hydraulic conductivity of 1.0×10^{-4} cm/s was assumed. We assumed a ratio of $k_h / k_v \sim 4$ for the embankment fill.

7.3.4 Steady State Seepage Analysis Results

Figures 8-1 through 8-4 show the contours of pressure head output from SEEP/W for Reach A through Reach D. The maximum total gradient for each cross section was taken from the largest contour near the landside toe.

The maximum gradient for each section is summarized in Table 2 for the hypothetical steady-state condition. Even with a steady state condition, the exit gradient is expected to be below 0.5 for all reaches.

7.3.5 Transient Seepage Analysis

Steady-state seepage is commonly performed as a conservative upper limit on seepage potential. However, for the Stevens Creek Levees the duration of any flood is expected to be less than 24 hours, and the likelihood of seepage reaching a steady-state condition is unlikely. To check the degree of conservatism in the assumption of stead-state seepage being reached, we performed one transient seepage analysis; we selected Section B as appearing reasonably representative of all of the levees.

As flood waters rise in the channel, water pressures on the faces of the levees will increase and seepage will progress into the levee. Given enough time, the seepage will pass through the levee and exit near the toe. Depending on how long the flood water is elevated, seepage may or may not proceed all the way through the levee. Transient analyses can evaluate the rate at which seepage progresses through the levees in response to transient flood water levels.

Using the 100-year water surface elevation hydrograph provided by S&W, the water elevations with time were set as boundary conditions in the levee channel. For conservatism, in this transient analysis, we increased permeabilities by one order of magnitude above the values indicated in Table 1. We then performed two types of transient seepage:

1) We applied the hydrograph (shown in Appendix F) to the Cross Section B to evaluate seepage through the levee. As can be seen in Figure 9-2, seepage traveled about 20 feet from the inboard toe into the embankment.

2) We instantaneously raised the waterside water level from the normal stream level to the level of the 100-year flood and kept it there for an indefinite period of time. In this analysis, seepage finally emerged from the downstream toe after about 7 days.

These analyses both is provide further support for the conclusion that seepage is very unlikely to emerge from the embankment during a design level flood that only lasts a matter of hours.

7.4 SLOPE STABILITY ANALYSIS

The computer program SLOPE/W (GeoSlope, Ltd., 2007) utilizes two-dimensional, force and moment limit-equilibrium methods to calculate factors of safety along specified slip surfaces through an embankment. Spencer's method, which satisfies both force and moment equilibrium and is restricted to a constant inter-slice force function, was used for all analyses. By specifying material zones delineated by strength functions, SLOPE/W estimates the normal stresses acting on a potential slip surface to calculate the shear strength along the surface and compares the strength with estimated (gravity-induced) driving stresses to calculate a factor of safety.

The same cross sections and soil profiles selected for the seepage analysis were used for the slope stability analysis. Slope "failure" may be loosely defined as anything from small displacements of portions of the embankment materials requiring additional maintenance, to large displacements of levee sections causing a breach of the system and requiring major repair.

Based on available data of cross sections and observations made during our field reconnaissance, existing slopes at the site appear to have maximum heights of approximately 15 feet on the waterside and 13 feet on the landside, with typical inclinations of 2:1 (horizontal: vertical) and crest widths of 10 ft.

Our levee stability analysis generally followed the procedure described in USACE EM 1110-2-1913. Four loading conditions were considered for each of the four reaches/cross sections:

- Existing Condition
- Steady State Seepage Condition
- Rapid Drawdown Condition
- Earthquake Condition

Existing condition is discussed in Section 7.4.1. Steady state seepage condition is discussed in Section 7.4.2. Rapid drawdown condition is discussed in Section 7.4.3. Earthquake condition is discussed in Section 7.4.4. Section 7.4.5 discusses the stability of the flood wall on the east bank from the geotechnical perspective.

7.4.1 Existing Condition

The existing condition considers a situation where the water level in the creek and ground water level are approximately 4 ft above the channel invert. This situation is based on the ground water measurements from the available boring logs and CPT data, and the field observation during our site reconnaissance. The analyses of slope stability for the existing condition were performed using effective stress strength parameters.

Seven cases were analyzed, including both waterside and landside of all cross sections except for the landside of Cross Section A where the landside ground surface is horizontal. The factors of safety are summarized in Table 2. It can be noted that the factors of safety of levee slope stability analyzed for the existing condition are satisfactory and ranging from 2.9 to 4.1. Figures 10-1 through 10-7 illustrate the failure surface, factor of safety contour, and model set-up for each case.

7.4.2 Steady State Seepage Condition

The steady state seepage condition considers a situation where flood water level is sustained long enough to create steady-state flow through the levee, increasing pore pressures within the levee and foundation materials, thereby decreasing the strength of the levee and levee foundation materials. We note that the steady state seepage assumption, though consistent with the state of the practice, is conservative for the levee and levee foundation conditions identified for our study. Locations of phreatic surfaces were taken directly from the results of the steady-state conditions developed in the seepage analysis using the design flood level. As defined in the seepage analysis, the design flood level is approximately at 4 ft below the levee crest. The stability computations were performed using effective stress strength parameters. Due to the water pressure against the waterside slope of the levee, the waterside of the levee is deemed more stable than existing condition. Therefore, only landside of the cross section B, C and D were analyzed for the slope stability under steady state conditions.

The factors of safety of these three cases were summarized in Table 2. The results indicate that the levees under steady state seepage condition are stable, with factors of safety ranging from 2.7 to 3.5. Figures 11-1 through 11-3 illustrate the failure surface, factor of safety contour, and model set-up for each case.

7.4.3 Rapid Drawdown Condition

The rapid drawdown loading condition considers a situation where flood water level are sustained long enough to create steady-state flow through the levee, and then water levels are quickly lowered, reducing the buttressing effect of the water loads on the waterside slope, thereby increasing the driving stresses on the potential failure surface. It was conservatively assumed that drawdown would be sufficiently fast that no drainage would occur in materials with low permeability. Effective stress strength parameters were used with a conservatively-drawn-down phreatic surface located at the ground surface on the waterside slope up to the elevation of the maximum flood level, and within the levee the steady state seepage phreatic surface was used.

Only the watersides of the levees were analyzed for the four cross sections for rapid drawdown condition. The landsides of the levees will not be less stable than for the steady state seepage condition, and therefore no additional analysis is necessary.

The factors of safety of these four cases are summarized in Table 2. The results show that the levee under rapid drawdown condition is stable, with factors of safety ranging from 2.9 to 3.5. Figures 12-1 through 12-4 illustrate the failure surface, factor of safety contours, and model configuration for each rapid drawdown case.

7.4.4 Seismic Stability Analysis

SLOPE/W provides an option to specify the resisting shear stress along the potential sliding surface when performing a pseudo-static analysis with an applied seismic coefficient. In this case, SLOPE/W first performs an analysis for a particular slip surface without applying the seismic force, and computes the shear resistance at the base of each slice. Next, the shear resistance is kept constant based on the initial confining stress prior to application of the seismic force. Then, the analysis is repeated for the same slip surface but including the seismic force. This option (where the undrained strength during earthquake loading is specified based on the confining stress prior to the earthquake) is used in this study.

The seismic stability of the levees was evaluated in two stages. For the initial stage we performed a pseudo-static seismic screening in which a seismic coefficient of 0.15 was applied, as suggested by Seed (1979) for a magnitude 8.25 earthquake. We conservatively reduced the undrained strength by 20 percent for soils that were thought to have a potential for seismic softening, as suggested by Makdisi and Seed (1978). Seed indicates that slopes with a resulting pseudo-static safety factor of at least 1.15 for full strengths or at least 1.00 for reduced strengths will have deformations less than about 3 feet.

For this initial screening, computed factors of safety for both waterside and landside of the levee were in the range of 1.6 to 2.0 (see Table 2). These screening cases are presented on Figures 13-1 through 13-7. The pseudo-static screening analyses of slope stability indicate that the slopes of the levees have factors of safety greater than the minimum value 1.00 for reduced strengths (see Table 2). These screening results suggest that the permanent displacements induced by the earthquake will not be larger than about 3 feet (Seed 1979).

Although not necessary to meet NFIP certification requirements, we considered it appropriate to further estimate the earthquake-induced deformations. For this second stage of analysis, permanent deformations were estimated using the yield acceleration concept proposed by Newmark (1965) and simplified by Makdisi and Seed (1978). The results of these additional runs are presented on Figures 14-1 through 14-7.

The procedure used to estimate permanent deformation includes the following steps.

- A yield acceleration, ky, at which a potential sliding surface would develop a factor of safety of 1.0, was determined using limit-equilibrium, pseudo-static slope stability methods. The yield acceleration depends on slope geometry, phreatic surface conditions, undrained shear strength of the slope material (and/or the residual strength of the liquefied zones), and the location of the potential sliding surface. The soil strength properties were reduced by 20% from the static undrained conditions, therefore the yield accelerations found in these analyses are considered to be conservative.

- The peak ground acceleration for a 475-year return period in alluvium was estimated from the CGS Probabilistic Seismic Hazards Map, 2002, to be 0.48g. With relatively low levees such as those along Stevens Creek, there is very little amplification of ground motion from the foundation to the crest of the levees; conversely, average accelerations within a sliding mass will not be much less than the PGA. Therefore we consider it appropriate to use the PGA directly for the seismic coefficient.

- For a specified potential sliding mass, the seismic coefficient time history of that mass is compared with the yield acceleration, ky. When the seismic coefficient exceeds the yield acceleration, down slope movement will occur along the direction of the assumed failure plane. The movement will decelerate and will stop after the level of induced acceleration drops below the yield acceleration, and the relative velocity of the sliding mass will drop to zero. The magnitude of the accumulated permanent displacement can be calculated by double-integrating the seismic coefficient time history where it exceeds the yield acceleration. However, to simplify this calculation, it is normal to use simplified design charts developed by Makdisi and Seed (1978) that relate the yield coefficient to the peak horizontal acceleration and the predicted earthquake magnitude.

The yield acceleration, ky, was calculated using limit-equilibrium stability analyses, as described above. The estimated ky values, factors of safety, and expected deformations for the waterside and landside slopes of Cross Sections A, B, C and D are summarized in Table 2. The computed lateral deformations are from about 3 inches to 32 inches. This means that, under seismic loading such as could occur during a major earthquake on a nearby fault, the slopes may be expected to deform somewhat. This would result in some loss of freeboard, with the magnitude of freeboard loss being on the order of one third to one half of the anticipated lateral deformations. In addition, some cracking of the embankment could occur.

Because of the long return period for a major earthquake, it is extremely unlikely that a flood will occur concurrently with a major earthquake. Similarly it is also unlikely that a major flood (i.e., with a long return interval) would occur in the months following the earthquake.

7.4.5 Flood Wall Stability Evaluation

The concrete flood wall on the east bank of the Stevens Creek Levee extends from approximately Levee Station 126+80 to approximately Levee Station 142+50. The height of the flood wall ranges from about 1 ft (near levee station 126+80) to about 8 ft (near Levee Station 142+50). The cross section selected for slope stability analysis within this reach is at River Station 114+96, which corresponds to approximately Levee Station 139+23. The height of the wall is approximately 8 ft. The thickness of the wall is about 8 inch. Based on the construction plan, the embedded depth of the wall when it is full height is 8.5 ft; we did not field-verify the wall embedment depths. The distance from the wall to the edge of the levee crest is about 12 ft. The global stability of the levee for this particular cross section was analyzed using SLOPE/W as discussed in prior sections. We only discuss the flood wall stability under flood conditions in this section. In other words, we evaluated whether the soil

around the flood wall will provide sufficient passive resistance (or have enough strength) to prevent the flood wall from a rotational failure. The wall responses (deflection, moment, shear force) are included as part of the output of the analysis. They are calculated based on the estimated earth and water pressures acting on the wall using classic soil mechanics procedures. However, evaluating the structural integrity of the flood wall is not within our scope of work.

We used the computer program CWALSHT to analyze the flood wall stability. CWALSHT was developed by the USACE for design and analysis of sheet-pile walls. The program uses classical soil mechanics procedures for determining the required depth of penetration of a new wall or assessing the factor of safety for an existing wall.

Soil (clay in this case) properties used in CWALSHT were consistent with what we used in SLOPE/W except that zero cohesion was assigned to the soil in order for the program to the run seepage condition. It is a conservative assumption. The interface friction angle between clay and the wall was selected at 18 degree based on NAVFAC DM-7.2 (1982). A Modulus of Elasticity of 3.8×10^6 psi was used for the concrete wall. The moment of inertia of the wall was calculated as 512 in^4/ft.

A seepage analysis of the cantilever wall system was performed (see Figure 8-1).

Based on the flood hydrograph provided by S&W, the design flood water level was set at elevation 23 ft, which is about 4 ft below the top of the wall. Clay is typically considered to have a relatively low impermeability; we assumed that the groundwater level on the landside will not change significantly in response to under-seepage beneath the wall, but will remain at about elevation 11 ft. Under this condition, the analysis results showed soil passive resistance factor of safety of 2.2 using effective stress (drained) strength parameters, and greater than 3.0 for undrained strengths. The maximum bending moment of the wall is 2,812 lb-ft.

To evaluate the sensitivity of the analysis to the groundwater level on the landside of the wall, we considered how the safety factor would be effected if animal burrows, cracks on the levee, or other conditions, resulted in the groundwater rising on the landside to ground surface at approximately elevation 19 ft was analyzed. The results indicate that under this condition, the factor of safety of soil passive resistance was 1.32 using effective stress (drained) strength parameters, and greater than 3.0 for undrained strengths. The maximum calculated seepage gradient was 0.235. The maximum bending moment of the wall was 2,802 lb-ft.

Based on our analysis, it is our opinion that the concrete flood wall will be stable under flood condition from the geotechnical perspective and excessive wall rotation due to surrounding soil failure is unlikely. We did not evaluate the structural integrity or design of the flood wall.

7.5 SETTLEMENT EVALUATION

7.5.1 Static Settlement

Based on the laboratory consolidation tests, the upper clays underlying the site are well over-consolidated, suggesting that consolidation settlement should largely happen within a short time after construction (e.g., not more than a couple of years). In addition, the current surveyed elevations of the crest suggest that the current crest elevation is slightly higher than the construction grades. Based on our review of the data and discussion with the District, it is our interpretation that this means there has been very little settlement since construction activities in the 1980. In our opinion, it is likely that future settlements will not be greater than past settlements.

7.5.2 Seismic Settlement

As described above in Section 7.1, earthquake-induced settlement is not expected to exceed more than about 1-2 inches due to densification of underlying sand layers.

8.0 CONCLUSIONS

Based on the results of our geotechnical investigation, it is our opinion that the Stevens Creek Levees meet the geotechnical criteria set forth in 44 CFR Section 65.10 of the National Flood Insurance Program and will remain stable under the loading conditions described in USACE Design Manual EM 1110-2-1913. A summary of each of the geotechnical criteria is provided below.

8.1 EROSION OF LEVEE EMBANKMENT DURING BASE FLOOD

It is our understanding that the levee experienced a flood on February 3, 1998, with an estimated regional return interval of about 80 years. We are not aware of any significant erosional damage caused by that event. In addition, the levee slopes generally consist of clayey soils that are generally well vegetated with grasses so that they are expected to be generally resistant to erosion. We recommend that the project levees be inspected on a regular basis for any signs of increased erosion. The levees should also be inspected for signs of erosion following any major flood events.

8.2 LEVEE EMBANKMENT STABILITY

Detailed analyses were performed to evaluate the embankment stability for various loading conditions. The loading conditions considered during this study included: (1) existing

conditions; (2) steady state seepage; (3) earthquake loading from a 475-year event, and (4) rapid drawdown. For each static loading condition, the calculated minimum Factors of Safety met or exceeded the stability criteria as described in USACE Manual EM 1110-2-1913 Design and Construction of Levees.

8.2.1 Seismic Stability and Deformation

As presented in Section 7.4.4, we anticipate that a major earthquake could result in up to about 4 to 32 inches of seismic deformation occurring. Due to the conservatisms in our analyses, we anticipate that most of the levee will move less than about 4 to 6 inches, and that if these larger movements occur at all, they would be in localized areas. We also anticipate that the strains would be distributed over a broad shear zone. Based on geometric considerations, we anticipate that the amount of loss of freeboard would be about one third to one half of the lateral deformation, i.e., on the order of 2 to 16 inches.

However, as noted in Section 7.4.4, it is unlikely that a major flood will occur during or in the months following a major earthquake, and thus the chance of the levee being overtopped as a consequence of earthquake-induced deformations is very low. Furthermore, for the project levees, even if a major earthquake with a 475-year return interval led to loss of 16 inches of freeboard, the levees could still contain a 100-year flood.

Given the low risk of joint occurrence, it seems reasonable to certify the levee with the current geometry, and to monitor post-earthquake performance as part of the ongoing levee maintenance, performing post-earthquake repairs as necessary to restore the certified freeboard.

8.3 LEVEE FOUNDATION STABILITY

Analyses were performed to evaluate the static and seismic settlement potential of the subject levee. Based on these results, we estimate future static settlement of less than 1 inch. In addition, we estimate approximately 1 to 2 inches of settlement during a 475-year earthquake event due to seismic densification.

Although localized zones of the foundation may be subject to liquefaction, we calculate that the liquefied strengths will generally be similar to the conservative clay strengths we assumed for our analyses, so the stability will not be greatly affected by these local occurrences of liquefaction.

8.4 ENCROACHMENTS/MAINTENANCE

During our field inspections, we observed several minor encroachments, such as minor landscape terraces notched into the landside of the levee slope. We understand that these

encroachments were constructed without obtaining proper building permits. The presence of these encroachments on the levee stability is considered to be negligible and the were not incorporated into our stability analyses; in our opinion they will not decrease the calculated Factor of Safety below the stability criteria established in USACE Manual EM 1110-2-1913 Design and Construction of Levees.

There were numerous pipe penetrations, mostly consisting of drain pipes extending into the Stevens Creek channel from beyond the levees. Most were controlled against backflow with flap gates. In at least one case, the flap gate was buried in sediments and not operable. We did not observe evidence of geotechnical problems, such as seepage along these penetrations. However, the control gates should be monitored and maintained as appropriate to keep them functioning properly.

Furthermore, we observed vegetation and evidence of animal burrowing (mostly gophers and possibly some ground squirrels) in a number of locations, as documented in Appendix B. We recommend regularly scheduled maintenance of the subject levees to fill these holes and to remove the animals making them.

9.0 LIMITATIONS

In the performance of its professional services, AMEC, its employees, and its agents comply with the standards of care and skill ordinarily exercised by members of our profession practicing in the same or similar localities. No other warranty, either express or implied, is made or intended in connection with the work performed by us, or by the proposal for consulting or other services, or by the furnishing of oral or written reports or findings. In the event that conclusions or recommendations based on data in this report are made by others, such conclusions and recommendations are not our responsibility unless we have been given an opportunity to review, and we concur in writing with, such conclusions or recommendations.

In presenting data generated or collected by others, AMEC does not warrant or in any other way attest to the accuracy or applicability of the data. Locations of previous explorations by others have been approximated based upon figures provided in reproduced reports, and in many cases do not provide sufficient information or detail for accurate or precise locating of the explorations. As such, locations should be considered approximate.

The scope of work for this study did not include an environmental assessment or investigation for the presence of natural or man-made contaminants in the soil, bedrock, or groundwater. No inference of such conditions should be made from information contained in this report.

10.0 REFERENCES

Atwater, B.F., Hedel, C.W., and Helley, E.J., 1977. *Late Quaternary depositional history, Holocene sea-level changes, and vertical crustal movement in southern San Francisco Bay, CA*: U.S. Geological Survey Professional Paper 1014, 15 pp.

Bakun, W. H., 1999. "Seismic Activity of the San Francisco Bay Region": *Bulletin of the Seismological Society of America*, vol. 89, pp. 764-784.

Berloger, Long and Associates, 1981. *Geotechnical Investigation, Proposed Stevens Creek Flood Control Improvements, Central Expressway to San Francisco Bay, Santa Clara County, California*, consultant report prepared for the Santa Clara Valley Water District, December 8.

California Geological Survey, 2003. *Seismic Hazards Zones, Mountain View Quadrangle*; scale 1:24,000.

California Geological Survey, 2003, *Seismic Shaking Hazards in California, Based on the USGS/CGS Probabilistic Hazard Assessment Model, 2002 (revised April 2003).* http://redirect.conservation.ca.gov/cgs/rghm/pshamap/pshamain.html.

Carrier III, W. 2003. "Goodbye, Hazen; Hello, Kozeny-Carman." *Canadian Geotechnical Journal, The Canadian Geotechnical Society*.

Dibblee, T.W., Jr., 2007. *Geologic Map of the Palo Alto and Mountain View Quadrangles, Alameda, San Mateo, and Santa Clara Counties, California*: Dibble Geologic Map Center Map DF-350, scale 1:24,000.

Duncan, J.M., Horz, R.C., and Yang, T.L., 1989. *Shear Strength Correlations for Geotechnical Engineering*. Virginia Tech Department of Civil Engineering, Geotechnical Engineering.

Duncan, J.M., and Wright, S.G., 2005. *Soil Strength and Slope Stability*. Wiley.

Department of Water Resources, 2008. *Guidance Document for Geotechnical Analyses, Urban Levee Geotechnical Evaluations Program*, Contract 4600007418, Revision 6, March 2008, prepared by URS in association with GEI and Fugro.

Ellsworth, W.L., 1990. "Earthquake History 17611-1989", in Wallace, R.E., ed., *The San Andreas Fault System, California*: U.S. Geological Survey Professional Paper 1515, pp. 153-188.

GEO-SLOPE International, Ltd., 2007. GeoStudio 2007 (Version 7.13) software suite, including *Slope/W Software for Computing Factor of Safety of Earth and Rock Slopes*, and *Seep/W Software for Performing 2D Seepage Analysis*. Calgary, Alberta, Canada.

Hall, T.N., Wright, R.H., and Clahan, K.B., 1999. "Paleoseismic Studies of the San Francisco Peninsula segment of the San Andreas Fault Zone Near Woodside, California": *Journal of Geophysical Research*, vol. 104, No. B10, p. 23,215-23,236.

Hall, T.N., Wright, R.H., and Prentice, C.S., 2001. "Studies along the Peninsula Segment of the San Andreas Fault, San Mateo and Santa Clara Counties, California", in *Engineering Geology Practice in Northern California*, edited by H. Ferriz and R.

Anderson: California Geological Survey Bulletin 210 / Association of Engineering Geologists Special Publication 12, p. 193-209.

Helley, E.J., Lajoie, K.R., Spangle, W.E., and Blair, M.L., 1979. *Flatland Deposits of the San Francisco Bay Region, California - Their Geology and Engineering Properties, and their Importance to Comprehensive Planning*: U.S. Geological Survey Professional Paper 943, 88 pp.

Lawson, A.C., 1908. *The California earthquake of April 18, 1906; Report of the California State Earthquake Investigation Commission*: Carnegie Institute, Washington, D.C., Publication 87, vol. 1 and atlas, 451 pp.

Makdisi, F.M., and H.B. Seed, 1978. "Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations", *Journal of the Geotechnical Engineering Division*, Vol. 104, No. 7, July 1978, pp. 849-867

Page, B.M., 1989. "Coast Range Uplifts and Structural Valleys", in Wahrhaftig, C. and Sloan, D. (eds.) *Geology of the San Francisco and Vicinity, 28th International Geological Congress Field Trip Guidebook* T105, p. 30- 32.

Porter, Urqhart, McCreary, and O'Brien Consultants, 1958. Borings Logs, in Sheet 23, Details, Zone NW-1 (North West) Project 1, Stevens Creek Improvement, June 16, 1960.

Seed, H.B., 1979. "Considerations in the Earthquake-Resistant Design of Earth and Rockfill Dams": *Geotechnique*, v. 29, n. 3, p. 215-263.

Tinsley, J.C., Egan, J.A., Kayen, R.E., Bennett, M.J., Kropp,A., and Holzer, T.L. 1998. Appendix: "Maps and descriptions of liquefaction and associated effects", in *The Loma Prieta, California, Earthquake of October 17, 1989 – Liquefaction*, edited by T.L. Holzer: U.S. Geological Survey Professional Paper 1551-B, p. B287-B314.

Toppozada, T.R., and Borchardt, G., 1998. "Re-evaluation of the 1836 "Hayward fault" and the 1838 San Andreas Fault Earthquakes": *Bulletin of the Seismological Society of America*, vol. 88, pp. 140-159, February.

Toppozada, T.R., and Parke, D.L., 1982. *Areas Damaged by California Earthquakes, 1900-1949: Annual Technical Report—Fiscal Year 1981-1982*: California Division of Mines and Geology, Open File Report 82-17, 65 pp.

Toppozada, T.R., Real, C.R., and Parke, D.L., 1981. *Preparation of isoseismal maps and summaries of reported effects for pre-1900 California earthquakes: Annual Technical Report—Fiscal Year 1980-1981*: California Division of Mines and Geology, Open File Report 81-11 SAC, 182 pp.

Townley, S.D., and Allen M.W., 1939. "Descriptive catalog of earthquakes of the Pacific Coast of the United States 1769 to 1928:: *Bulletin of the Seismological Society of America*, vol. 29, no. 1, 297 pp.

U.S. Army Corps of Engineers (1990). *Engineer Manual 1110-1-1904, Engineering and Design, Settlement Analysis*, September 30

U.S. Army Corps of Engineers (2000). *Engineer Manual 1110-2-1913, Engineering and Design, Design and Construction of Levees*, April 30

U.S. Army Corps of Engineers, 2006. *CWALSHT for Windows* Computer Program. February.

U.S. Geological Survey, 2002. *2002 Interactive Deaggregations*, http://eqint.cr.usgs.gov/deaggint/2002/.

WCGEP: see 2007 Working Group on California Earthquake Probabilities

Witter, R.C., Knudsen, K.L., Sowers, J.M., Wentworth, C.M., Koehler, R.D., and Randolph, C.E., 2006. *Maps of Quaternary deposits and liquefaction susceptibility in the central San Francisco Bay Region, California*; United States Geological Survey Open-File Report 06-1037.

2007 Working Group on California Earthquake Probabilities (WGCEP), 2008. *The Uniform California Earthquake Rupture Forecast, Version 2 (UCERF 2)*: United States Geological Survey Open File Report 2007-1437 and California Geological Survey Special Report 203.

Youd, T.L., and Hoose, S.N., 1978. *Historic Ground Failures in Northern California Triggered by Earthquakes*: U.S. Geological Survey, Professional Paper 993, 177 p.

Youd, T.L., et al., 2001. "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils", *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 127, No. 10, American Society of Civil Engineers, October.

Zhang G., Robertson. P.K., Brachman R., 2002. "Estimating Liquefaction Induced Ground Settlements from the CPT", *Canadian Geotechnical Journal*, 39: pp 1169-1180

Zhang G., Robertson. P.K., Brachman R., 2004. "Estimating Liquefaction Induced Lateral Displacements using the SPT and "PT", ASCE, *Journal of Geotechnical & Geoenvironmental Engineering*, Vol. 130, No. 8, 861-871.

TABLES

TABLE 1: MATERIAL PROPERTIES

Stevens Creek Soil Properties							
Soil Type	Properties		Existing	Steady State	Drawdown	Seismic	Source
Fill (CL-CH, SC)	Unit Weight, γ_t	120 pcf					Average of all east borings
	Permeability, k_v	$1.0x10^{-4}$ cm/s ($3.3x10^{-6}$ ft/s)					Assumed from Fig. 7.6 in Holtz and Kovacs for very fine sand, mixture of sand, silt and clay, stratified clay deposits
	k_h/k_v	4					DWR (2008), page 3-4, Table 3-1 Permeability Values
	Undrained Shear Strength, s_u	1500 psf (CL-CH)				X	Estimated from correlation of strength properties from SPT data and CPT data
	Total Stress Cohesion, c	300 psf			X		Assumed same as effective strength
	Total Stress Friction Angle, ϕ	28°			X		Assumed same as effective strength
	Effective Cohesion, c'	300 psf	X	X	X		Table 5.8 in Duncan and Wright (2005)
	Effective Friction Angle, ϕ'	28°	X	X	X		Estimated from chart by Duncan et al. (1989) using average PI=45, and Table 5.8 in Duncan and Wright (2005)
Clay	Unit Weight, γ_t	127 pcf					Average of all east borings
	Permeability, k_v	$2.8x10^{-7}$ cm/s ($9.3x10^{-9}$ ft/s)					Average from consolidation tests B-1 and B-2 (same soil layer)
	k_h/k_v	10					DWR (2008), page 3-4, Table 3-1 Permeability Values
	Undrained Shear Strength, s_u	800 psf				X	Estimated from correlation of strength properties from SPT data and CPT data
	Total Stress Cohesion, c	600 psf			X		CU test B-2 and B-1
	Total Stress Friction Angle, ϕ	18°			X		CU test B-2 and B-1
	Effective Cohesion, c'	300 psf	X	X	X		CU test B-2 and B-1
	Effective Friction Angle, ϕ'	30°	X	X	X		CU test B-2 and B-1

TABLE 1: MATERIAL PROPERTIES

Stevens Creek Soil Properties							
Soil Type	Properties		Existing	Steady State	Drawdown	Seismic	Source
Sand	Unit Weight, γ_t	131 pcf					Average of all east borings
	Permeability, k_v	$1.2x10^{-2}$ cm/s ($4.0x10^{-4}$ ft/s)					Kozeny-Carman average value for sand layer
	k_h/k_v	4					DWR (2008), page 3-4, Table 3-1 Permeability Values
	Undrained Shear Strength, s_u	--					
	Total Stress Cohesion, c	0 psf			X	X	
	Total Stress Friction Angle, ϕ	35°			X	X	Estimated from correlation of strength properties from blow counts - Meyerhof (35) and from CPT data (35)
	Effective Cohesion, c'	0 psf					
	Effective Friction Angle, ϕ'	35°	X	X	X		Assume same as ϕ
Clay (deep)	Unit Weight, γ_t	129 pcf					Average of all east borings
	Permeability, k_v	$2.8x10^{-7}$ cm/s ($9.3x10^{-9}$ ft/s)					Average from consolidation tests B-1 and B-2 (same soil layer)
	k_h/k_v	10					DWR (2008), page 3-4, Table 3-1 Permeability Values
	Undrained Shear Strength, s_u	1200 psf			X	X	Estimated from correlation of strength properties from SPT data and CPT data
	Total Stress Cohesion, c	--					
	Total Stress Friction Angle, ϕ	--					
	Effective Cohesion, c'	300 psf	X	X	X		Assumed value, considered CU test B-2 and B-1 for upper layer of clay
	Effective Friction Angle, ϕ'	31°	X	X	X		Estimated from chart by Duncan et al. (1989) using average PI=20

TABLE 2: LEVEE SLOPE STABILITY ANALYSIS RESULTS

Analysis Condition	Strength Type	Required minimum Factor of Safety	Computed Factor of Safety							
			Reach A		Reach B		Reach C		Reach D	
			W	L	W	L	W	L	W	L
Existing	Drained	1.3	3.70	--	3.01	3.26	2.90	4.07	3.13	3.63
Steady-State Seepage	Drained	1.4[1]	--	--	--	2.76	--	3.53	--	3.11
Rapid Drawdown	3-stage	1.2[1]	3.06	--	2.33	--	2.27	--	2.47	--
Pseudo-Static Screening	Undrained	1.00 - 1.15[2]	1.43	--	1.28	1.51	1.20	1.62	1.28	1.61
			Computed Seismic Yield Coefficient							
Seismic Yield Acceleration	Undrained	n/a	1.01 (0.24g)	--	1.00 (0.225g)	1.00 (0.26g)	1.00 (0.208g)	1.00 (0.278g)	1.01 (0.23g)	1.00 (0.285g)
			Maximum Computed Exit Gradient							
Maximum Exit Gradient	Steady-State Seepage		0.0		0.35		0.15		0.30	
			Range of Computed Seismic Deformation[2] (inches)							
Seismic Deformation			5-21	--	6-27	4-16	8-32	3-11	6-26	3-10
			Range of Computed Seismically Induced Settlement[3] (inches)							
Seismic Settlement			0.26 - 0.4		0.06 - 1.2		0.04 - 0.15		0.05 - 0.58	

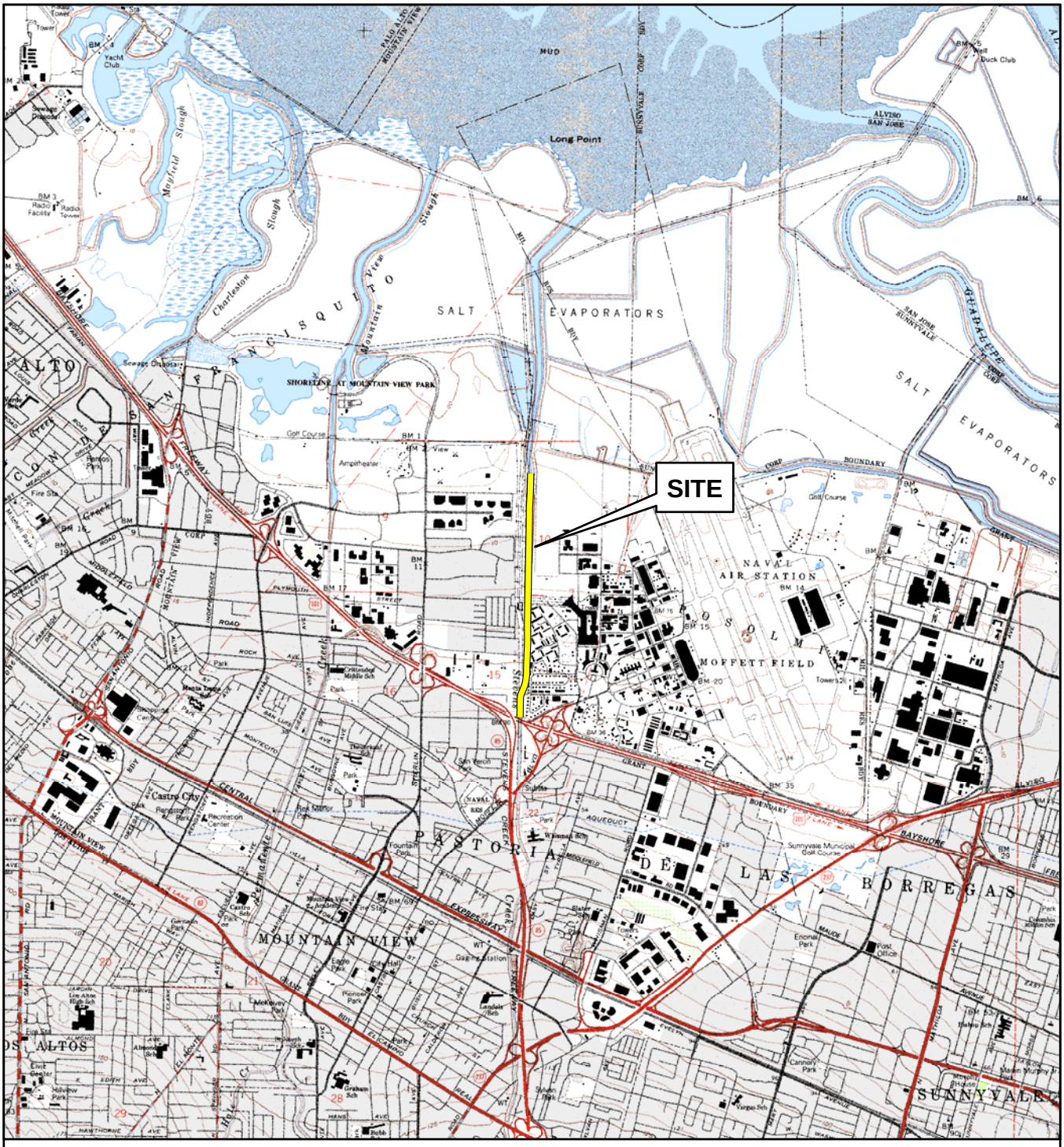
[1] USACE, 2000

[2] Makdisi and Seed (1978) & Seed (1979)

[3] CLiq analysis

W denotes waterside. L denotes landside.

FIGURES



Base map from USGS 7.5' Mountain View, California topographic quadrangle.



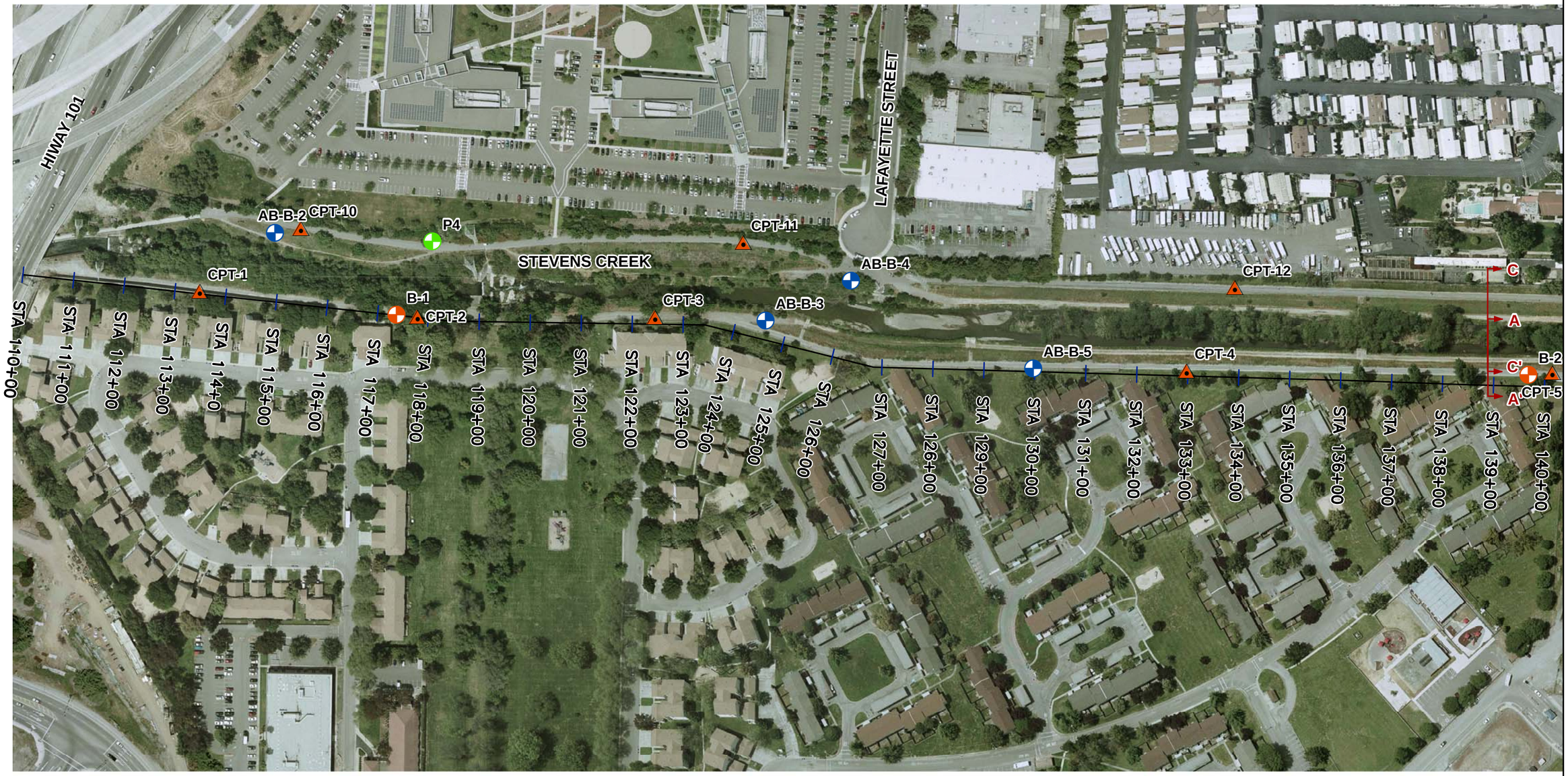
N

0 2,000 4,000 Feet

SITE LOCATION MAP
Stevens Creek Levees
FEMA Levee Certification
Mountain View, California

By: CED	Date: 2/23/2009	Project No. 13815.002
AMEC Geomatrix		Figure 1

File path: Q:\CorpStndrs\MASTERS\GDS\Titleblocks\amec_geomatrix\arcgis\A_portrait.mxd



Explanation

- Approximate location of previous exploratory borings by Porter, Urqhart, McCreary, and O'Brien Consultants (1958)
- Approximate location of previous exploratory borings by Berloger (1981)
- Approximate location of exploratory borings by Pacific Geotechnical Engineering, this study
- Approximate location of exploratory cone penetration test (CPT) probes by Pacific Geotechnical Engineering, this study

N

0 100 200 Feet

LEVEE ALIGNMENTS AND
FIELD EXPLORATION MAP
SOUTHERN REACH
Stevens Creek Levees
FEMA Levee Certification
Mountain View, California

By: DMO	Date: 2/24/2009	Project No. 13815.002
AMEC Geomatrix		Figure 2

File path: S:\13800\13815\13815.002\task_0B_03\09_0220_flclapp_fig_2.mxd



Explanation

- Approximate location of previous exploratory borings by Porter, Urqhart, McCreary, and O'Brien Consultants (1958)
- Approximate location of previous exploratory borings by Berloger (1981)
- Approximate location of exploratory borings by Pacific Geotechnical Engineering, this study
- Approximate location of exploratory cone penetration test (CPT) probes by Pacific Geotechnical Engineering, this study

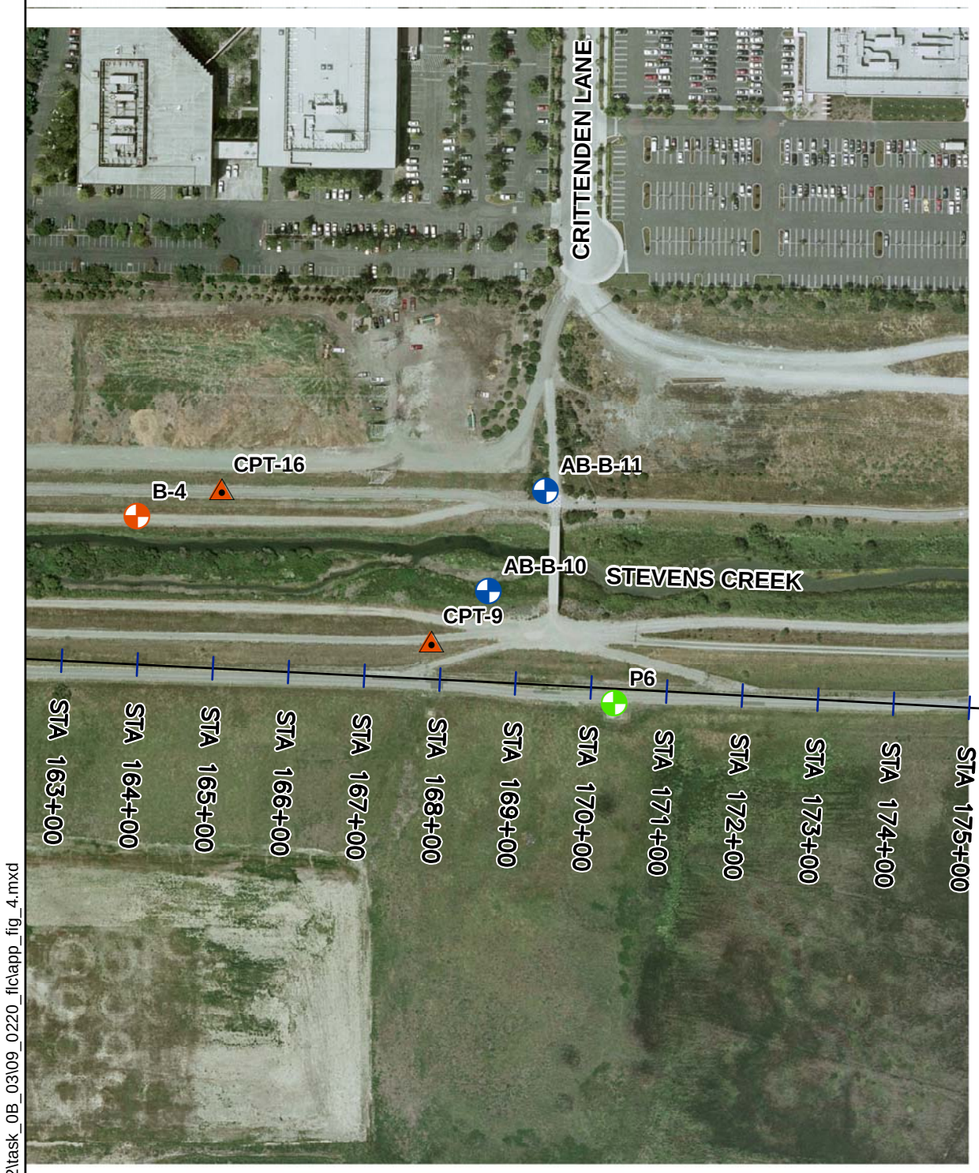
N

0 100 200 Feet

LEVEE ALIGNMENTS AND
FIELD EXPLORATION MAP
CENTRAL REACH
Stevens Creek Levees
FEMA Levee Certification
Mountain View, California

By: DMO	Date: 2/24/2009	Project No. 13815.002
AMEC Geomatrix		Figure 3

File path: S:\13800\13815\13815.002\task_0B_03\09_0220_fig\app_fig_3.mxd



Explanation

- Approximate location of previous exploratory borings by Porter, Urqhart, McCreary, and O'Brien Consultants (1958)
- Approximate location of previous exploratory borings by Berloger (1981)
- Approximate location of exploratory borings by Pacific Geotechnical Engineering, this study
- Approximate location of exploratory cone penetration test (CPT) probes by Pacific Geotechnical Engineering, this study

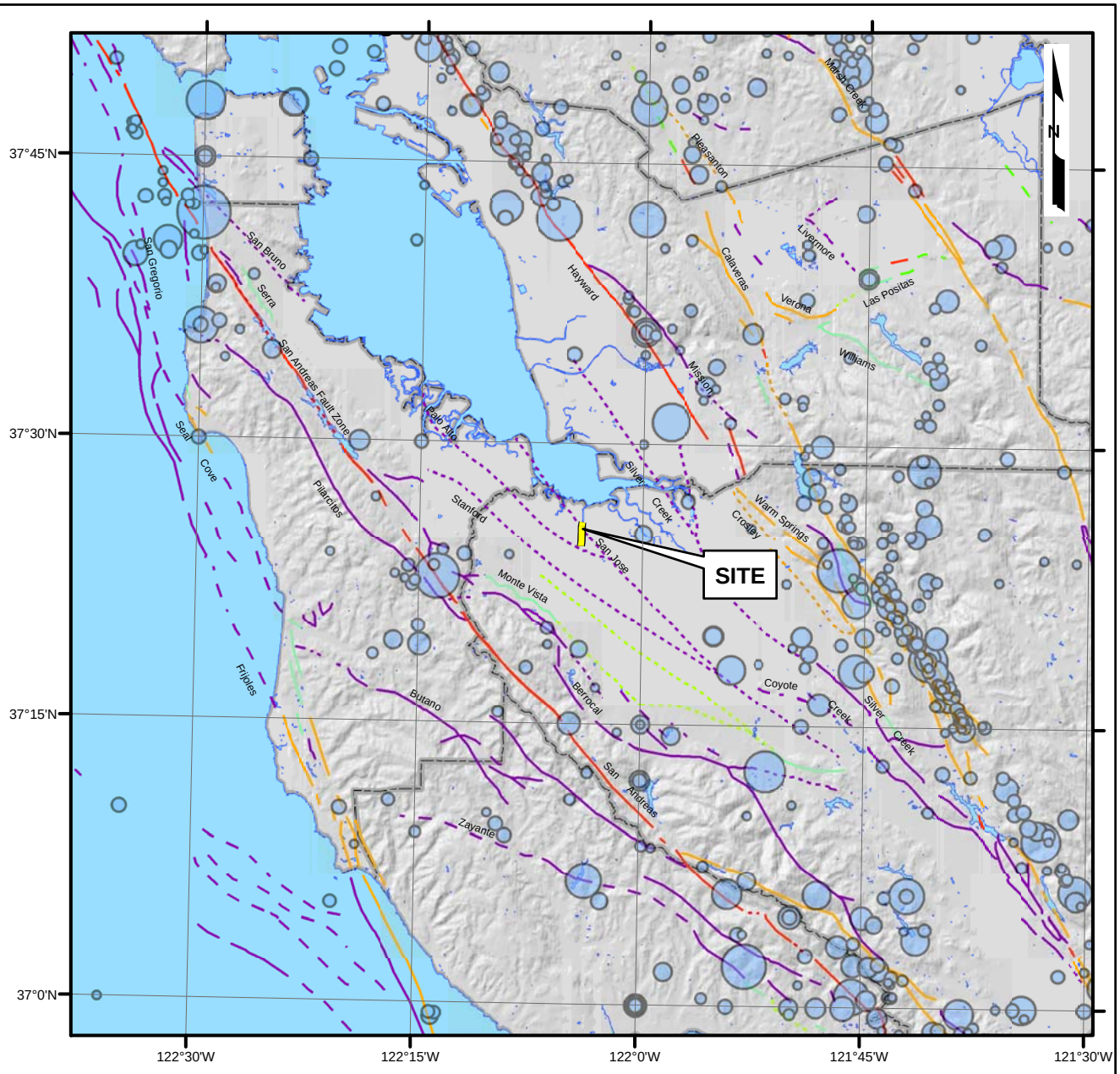
N

0 100 200 Feet

LEVEE ALIGNMENTS AND
FIELD EXPLORATION MAP
NORTHERN REACH
Stevens Creek Levees
FEMA Levee Certification
Mountain View, California

By: DMO	Date: 2/24/2009	Project No. 13815.002
AMEC Geomatrix	Figure 4	

File path: S:\13800\13815\13815.002\task_0B_03\09_0220_flc\app_fig_4.mxd



Explanation

Fault traces on land shown as solid where well located, dashed where uncertain or inferred, dotted where buried. Data from Jennings (1994).

- Faults with historic surface rupture/creep
- Faults that displace Holocene (~11 ka) or latest Pleistocene (~20 Ka) deposits or geomorphic surfaces.
- Faults that displace Late Quaternary (~ 780 ka) deposits or geomorphic surfaces.
- Quaternary faults, undifferentiated (1.8 Ma).

Independant Seismicity (1769-2008)

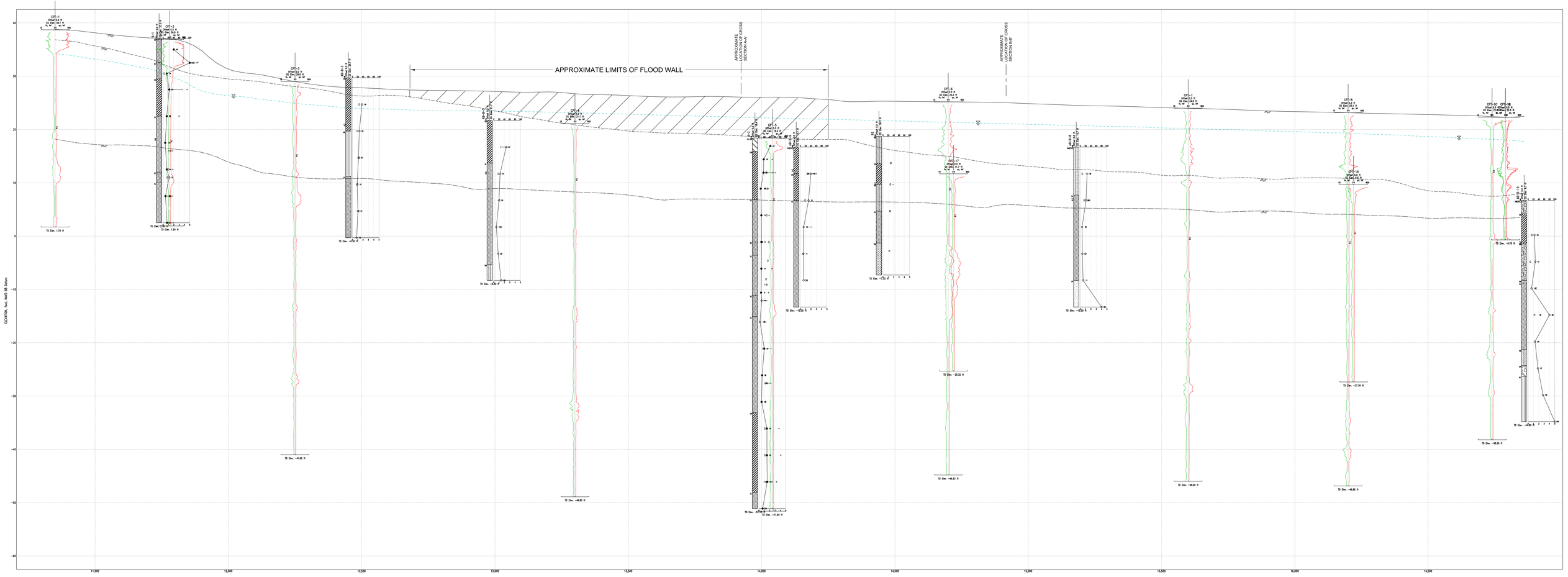
- 3.0
- 4.0
- 5.0
- 6.0
- 7.0

0 5 10 Miles

REGIONAL FAULT AND HISTORIC SEISMICITY MAP
Stevens Creek Levees
FEMA Levee Certification
Mountain View, California

By: DMO	Date: 2/20/2009	Project No. 13815.002
AMEC Geomatrix		Figure 5

File path: S:\13800\13815\13815.002\task_0B_03\09_0220_flc_fig_02.mxd



Surfaces Legend
- CREST ELEVATION
- WATER SURFACE ELEVATION (PROJECTED FROM CHANNEL)
- NATURAL GROUND SURFACE ELEVATION (PROJECTED FROM LANDSIDE TOE)
- CHANNEL INVERT ELEVATION (PROJECTED FROM CHANNEL)

Boring Plot Legend
- Water Lel Cal
- CPT: ■ Actual (Modified, 0.62)
- Water Content, %
- P-No. %
- Su,Tsf: + Pocket Pen. × Torvane
- ▽ Water Level at Time of Drilling
- ▼ Water Level at End of Drilling

Vertical Scale = 5.00 ft/in.
Horizontal Scale = 100.00 ft/in.

Begin Line North 100.00 East 0.00
End Line North 100.00 East 17000.00

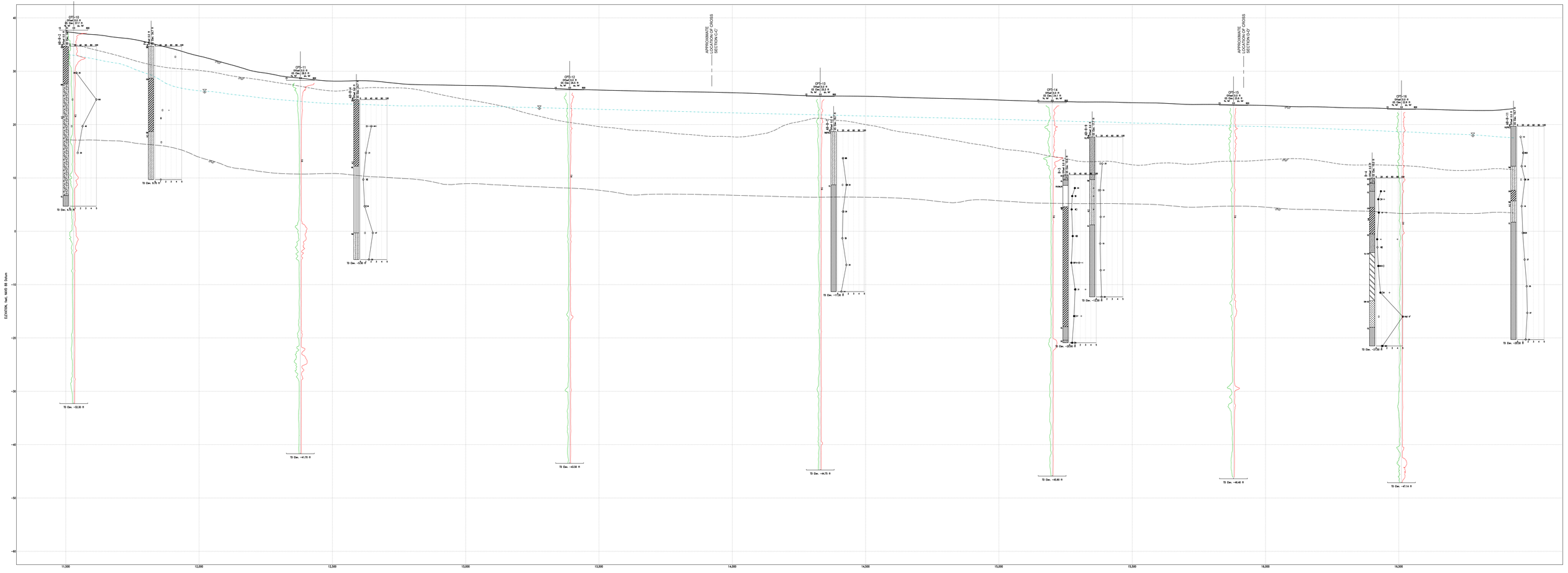
LIST OF BORING NUMBER AND OFFSET

CPT-5	0.2 In.	(20.0 ft)
CPT-17	0.2 In.	(20.0 ft)
CPT-18	0.2 In.	(20.0 ft)
CPT-9C	<0.3 In.	(<30.0 ft)

NOTES
1. PROFILE ELEVATIONS WERE PROVIDED BY SCHAAFF & WHEELER.
2. ELEVATIONS ARE APPROXIMATE.
3. BORING AND CPT LOCATIONS ARE SHOWN IN FIGURES 2 THROUGH 4. ELEVATIONS OF BORINGS AND CPTS WERE ESTIMATED BASED ON PROFILE ELEVATIONS.

East Bank Levee Profile
Stevens Creek Levees
FEMA Levee Certification
Mountain View, California

By: JH	Date: 02/03/06	Project: 1381S.005
		Figure: 6



Surfaces Legend

- CREST ELEVATION
- WATER SURFACE ELEVATION (PROJECTED FROM CHANNEL)
- NATURAL GROUND SURFACE ELEVATION (PROJECTED FROM LANDSIDE TOE)
- CHANNEL INVERT (PROJECTED FROM CHANNEL)

Boring Plot Legend

Vertical Scale = 5.00 ft/in.
Horizontal Scale = 100.00 ft/in.

Begin Line North: 100.00 East: 0.00
End Line North: 100.00 East: 17000.00

LIST OF BORING NUMBER AND OFFSET

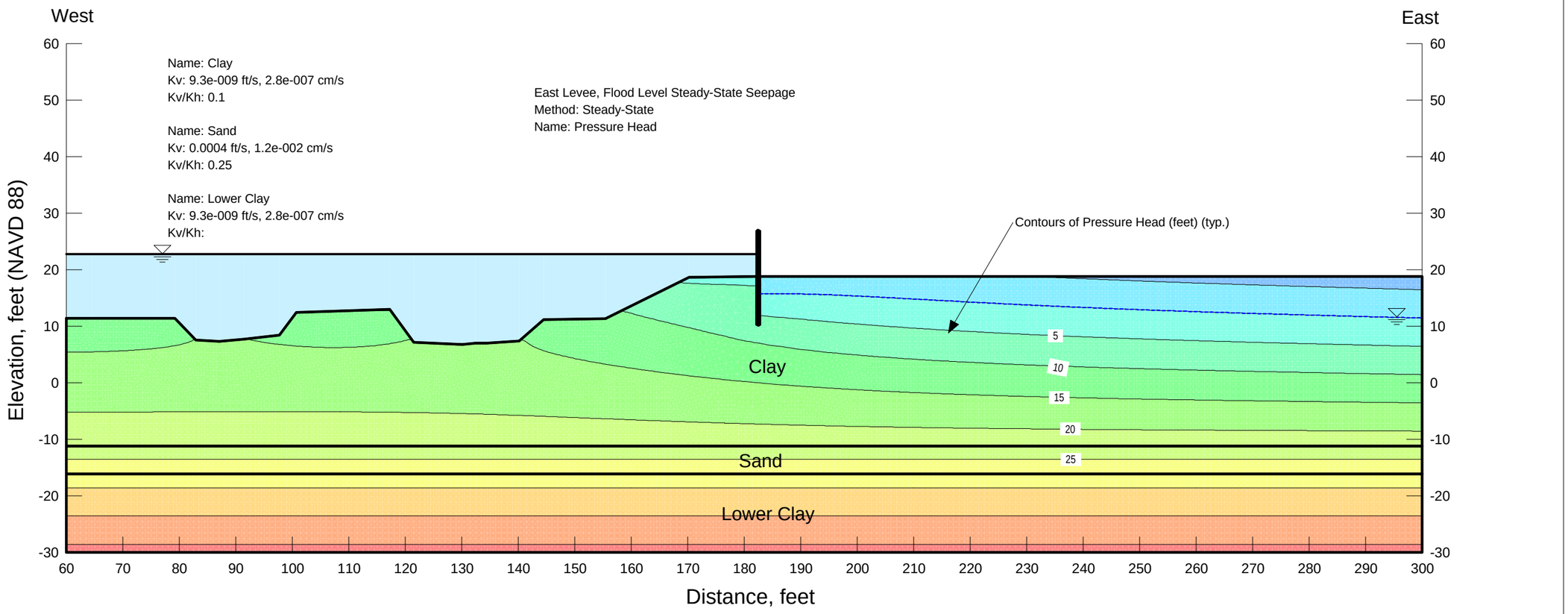
CPT-13	-0.5 in. (-50.0 ft)
B-3	+1 in. (+100.0 ft)
CPT-14	-1.5 in. (-150.0 ft)
B-2	+1 in. (+100.0 ft)

NOTES

1. PROFILE ELEVATIONS WERE PROVIDED BY SCHAAFF & WHEELER.
2. ELEVATIONS ARE APPROXIMATE.
3. BORING AND CPT LOCATIONS ARE SHOWN IN FIGURES 2 THROUGH 4. ELEVATIONS OF BORINGS AND CPTS WERE ESTIMATED BASED ON PROFILE ELEVATIONS.

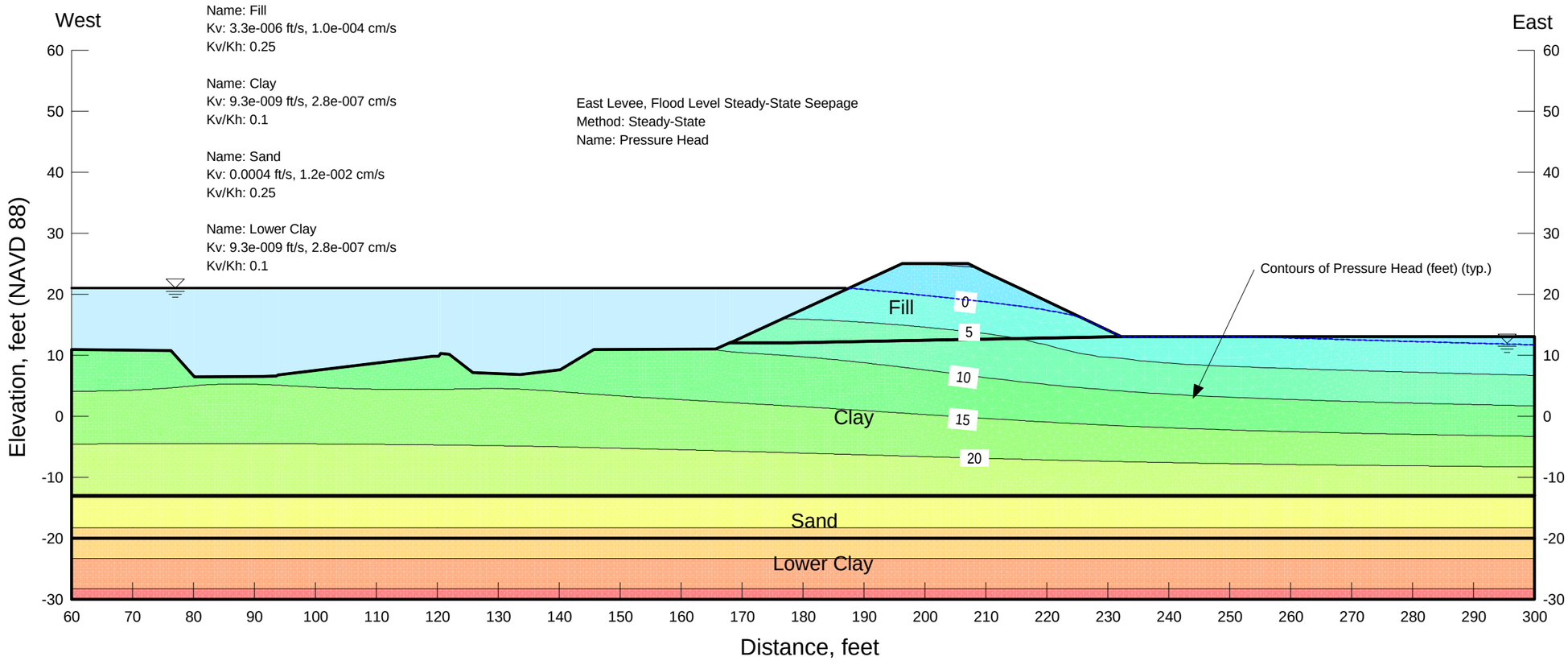
West Bank Levee Profile
Stevens Creek Levees
FEMA Levee Certification
Moutain View, California

By:	JH	Date: 02/03/09	Project: 1.3815.001
			Figure 7



amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Seepage Analysis Result** Flood Water Level Condition East Levee, Reach A Levee Station 139+23	**Figure** 8-1
		Task 0B.03	Analyzed D. Levulett			
		Date 2/27/2009	Checked			

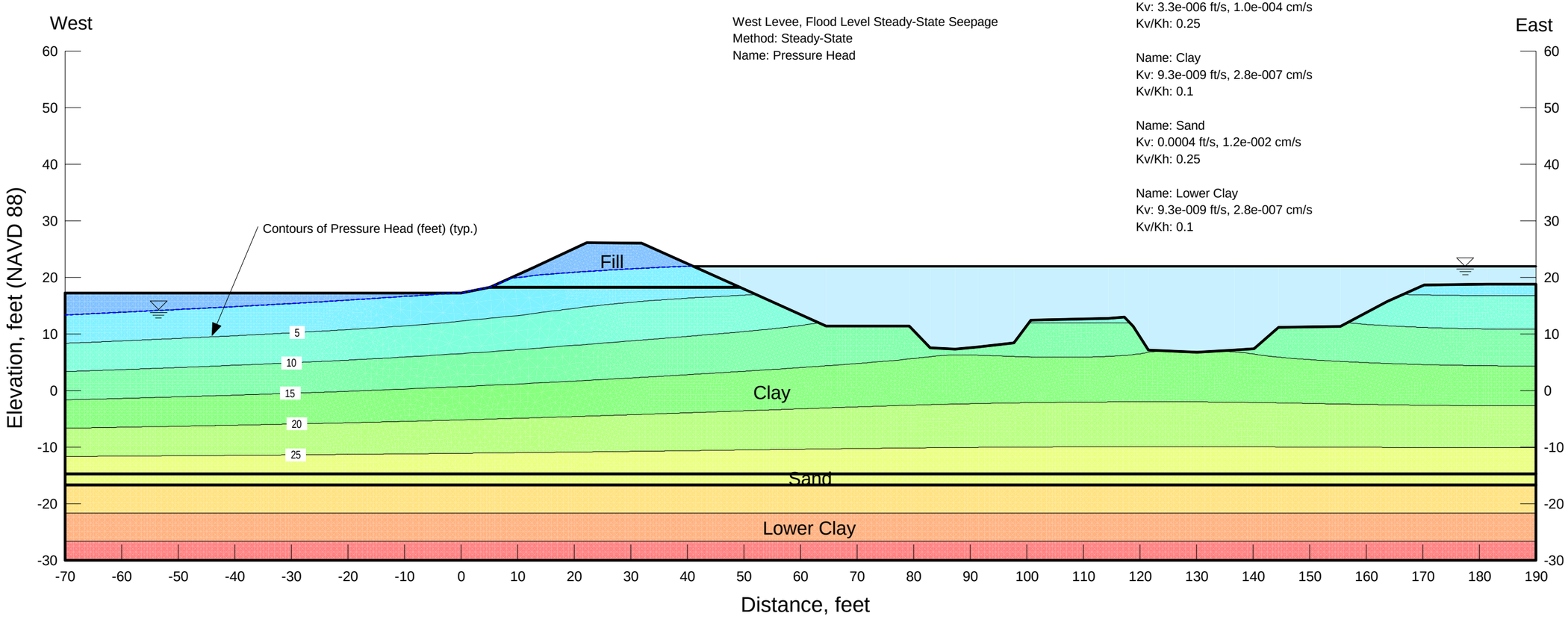
Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach A\East_A_RS_11496_SteadyState.gsz Date: 2/27/2009 Time: 2:12:16 PM Last Edited By: Levulett, Darcie



Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach B\East_B_RS_10502_SteadyState.gsz Date: 2/27/2009 Time: 1:55:38 PM Last Edited By: Levulett, Darcie

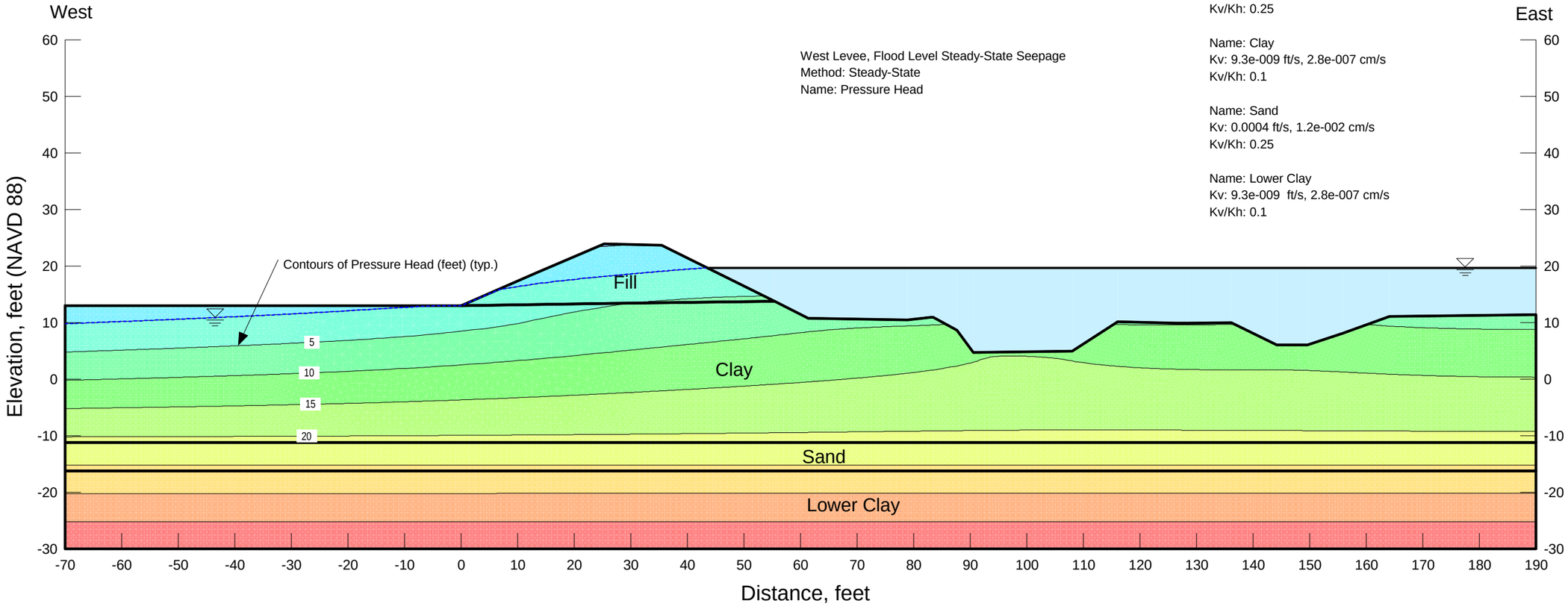
amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Seepage Analysis Result** Flood Water Level Condition East Levee, Reach B Levee Station 146+17	**Figure** 8-2
		Task 0B.03	Analyzed D. Levulett			
		Date 2/27/2009	Checked			

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach C\West_C_RS_13923_SteadyState.gsz Date: 2/27/2009 Time: 2:03:16 PM Last Edited By: Levulett, Darcie



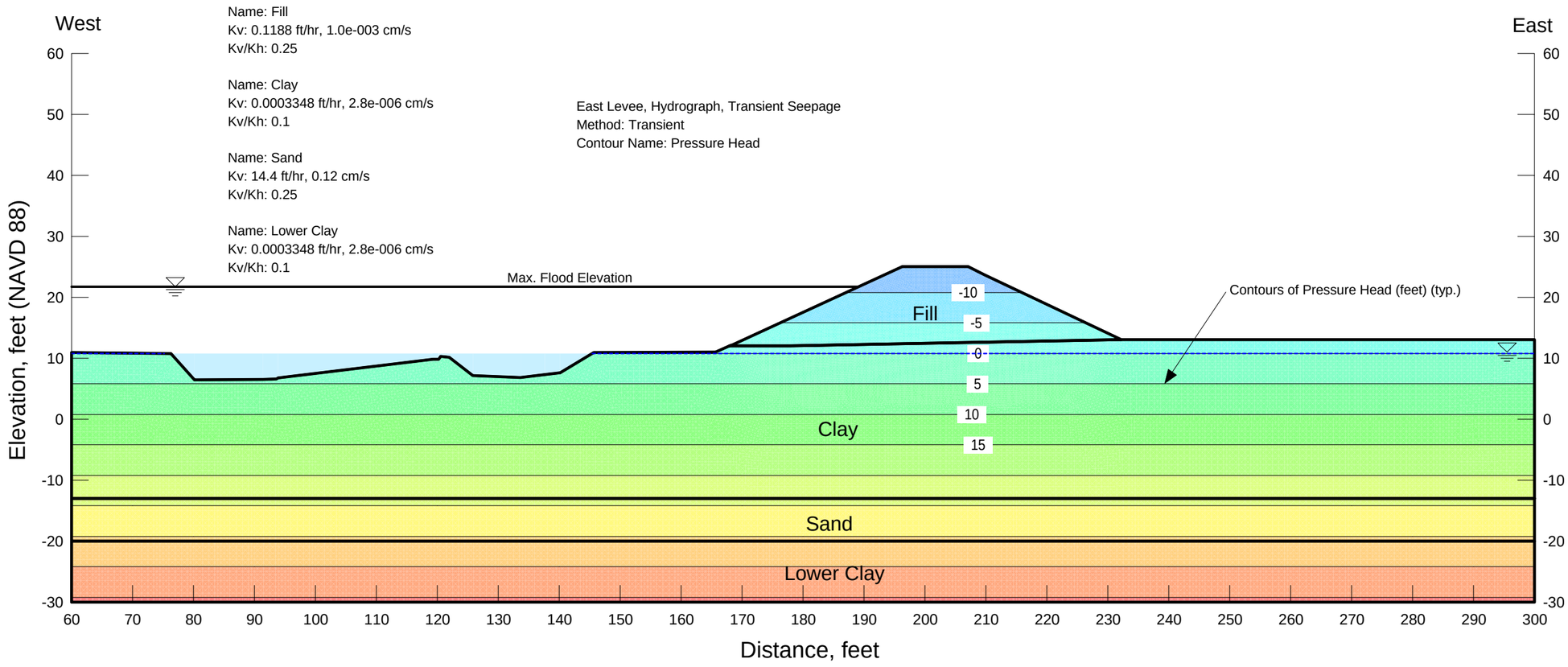
amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Seepage Analysis Result** Flood Water Level Condition West Levee, Reach C Levee Station 139+23	**Figure** 8-3
		Task 0B.03	Analyzed D. Levulett			
		Date 2/27/2009	Checked			

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach D\West_D_RS_9501_SteadyState.gsz Date: 4/29/2009 Time: 1:19:12 PM Last Edited By: Levulett, Darcie



amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Seepage Analysis Result** Flood Water Level Condition West Levee, Reach D Levee Station 159+18
		Task 0B.03	Analyzed D. Levulett		**Figure** 8-4
		Date 4/29/2009	Checked		

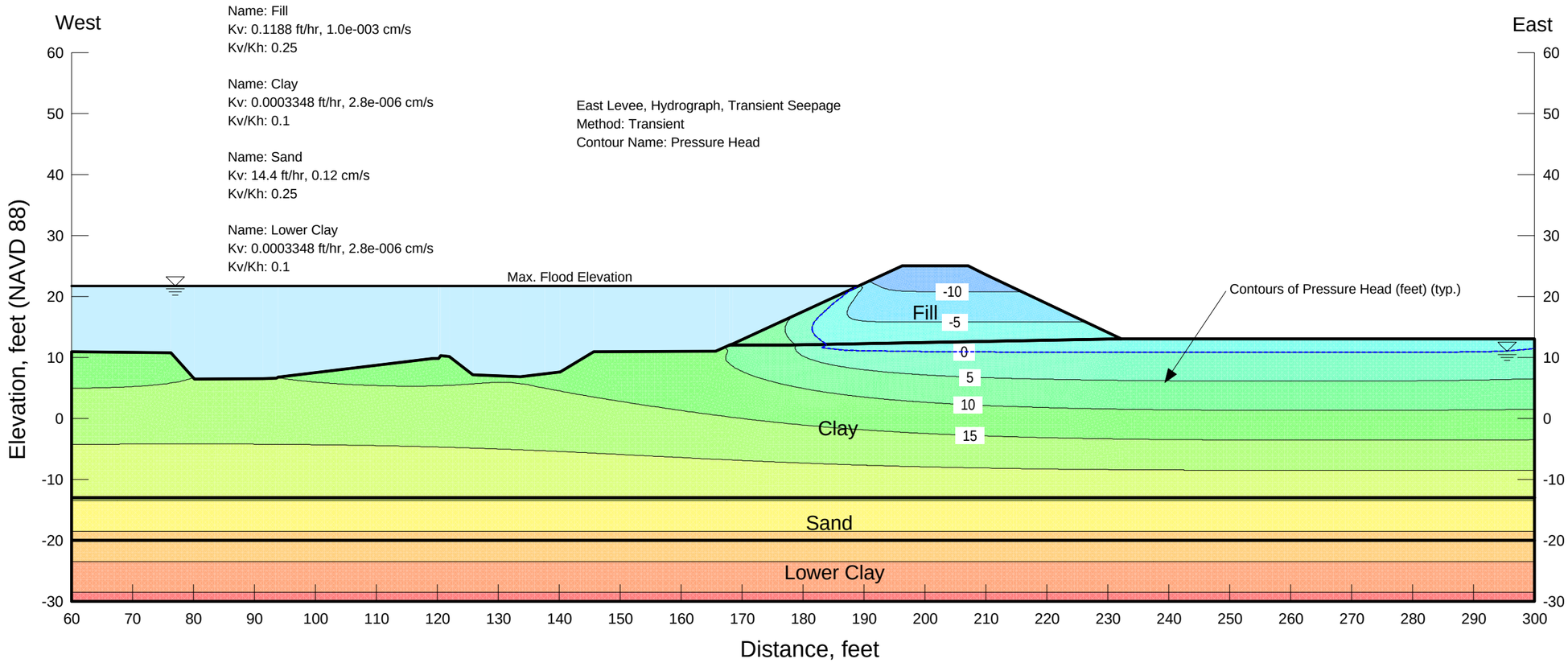
Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach B\East_B_RS_10502_Transient3.gsz Date: 3/3/2009 Time: 10:08:16 AM Last Edited By: Levulett, Darcie



Time = 0 hr (Pre-Flood)

amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Transient Seepage Analysis Result** Flood Water Level Condition East Levee, Reach B Levee Station 146+17	**Figure** 9-1
		Task 0B.03	Analyzed D. Levulett			
		Date 3/3/2009	Checked			

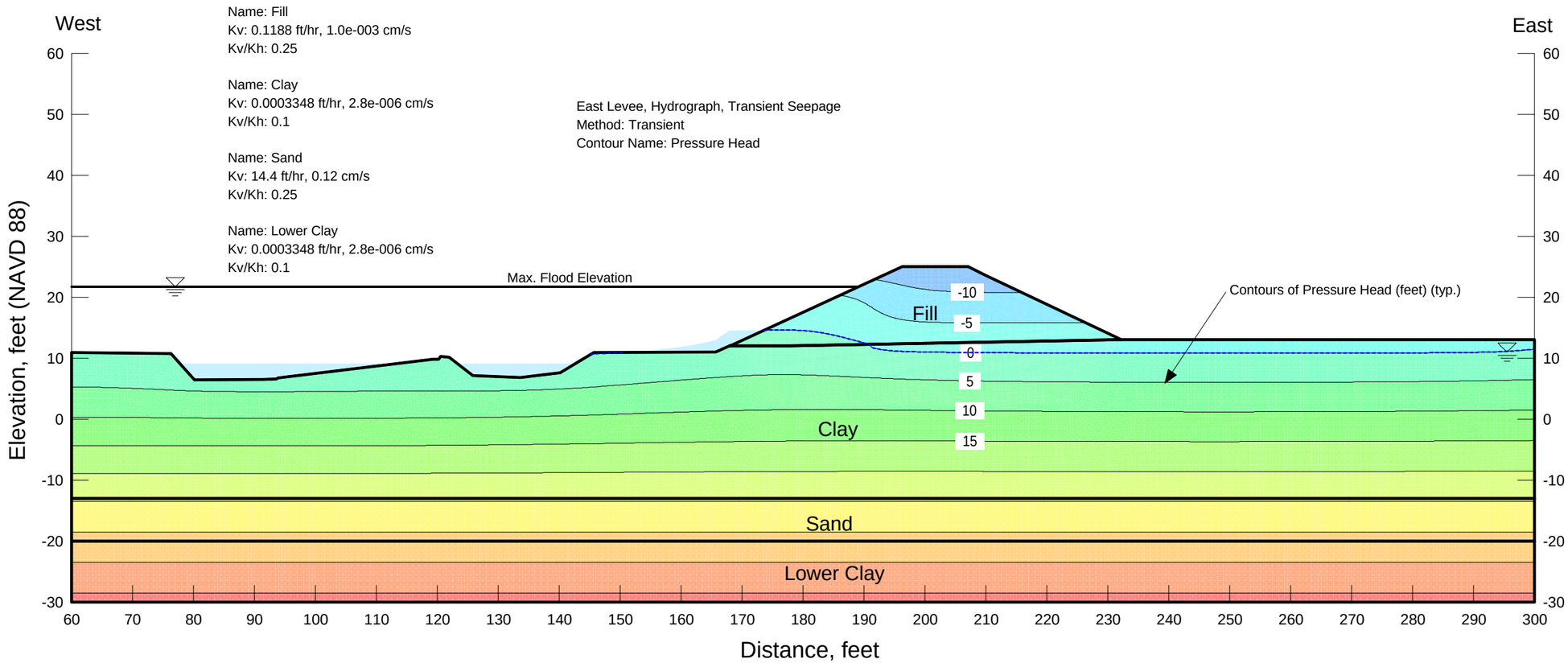
Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach B\East_B_RS_10502_Transient3.gsz Date: 3/3/2009 Time: 10:08:16 AM Last Edited By: Levulett, Darcie



Time = 14.4 hr (Max. Flood)

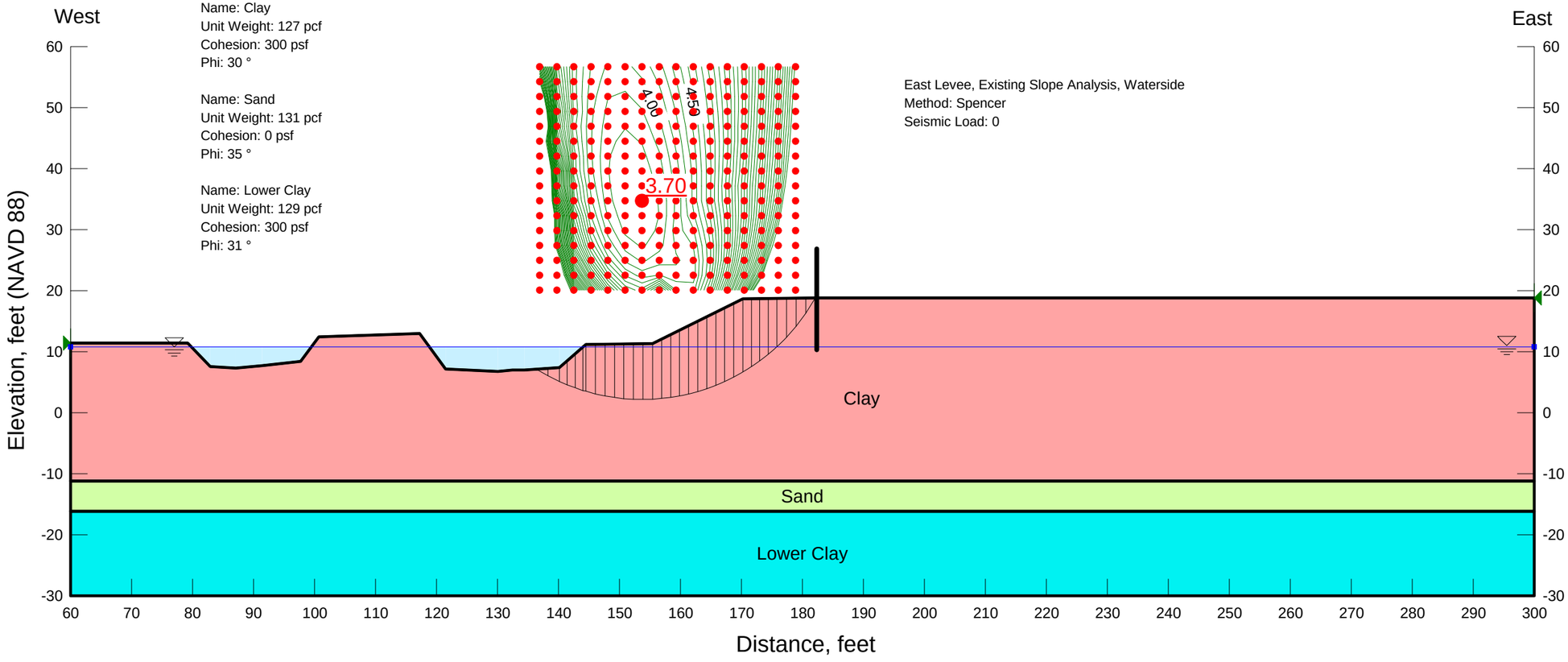
amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Transient Seepage Analysis Result** Flood Water Level Condition East Levee, Reach B Levee Station 146+17	**Figure** 9-2
		Task 0B.03	Analyzed D. Levulett			
		Date 3/3/2009	Checked			

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach B\East_B_RS_10502_Transient3.gsz Date: 3/3/2009 Time: 10:08:16 AM Last Edited By: Levulett, Darcie



Time = 24 hr (Flood Level = 9.13 feet)

amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Transient Seepage Analysis Result** Flood Water Level Condition East Levee, Reach B Levee Station 146+17	**Figure** 9-3
		Task 0B.03	Analyzed D. Levulett			
		Date 3/3/2009	Checked			



Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach A\East_A_RS_11496_Existing.gsz Date: 2/25/2009 Time: 1:14:10 PM Last Edited By: Levulett, Darcie

amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Existing Condition East Levee, Reach A Levee Station 139+23	**Figure** 10-1
		Task 0B.03	Analyzed D. Levulett			
		Date 2/25/2009	Checked PSK			

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach B\East_B_RS_10502_Existing.gsz Date: 2/18/2009 Time: 11:56:45 AM Last Edited By: Levulett, Darcie

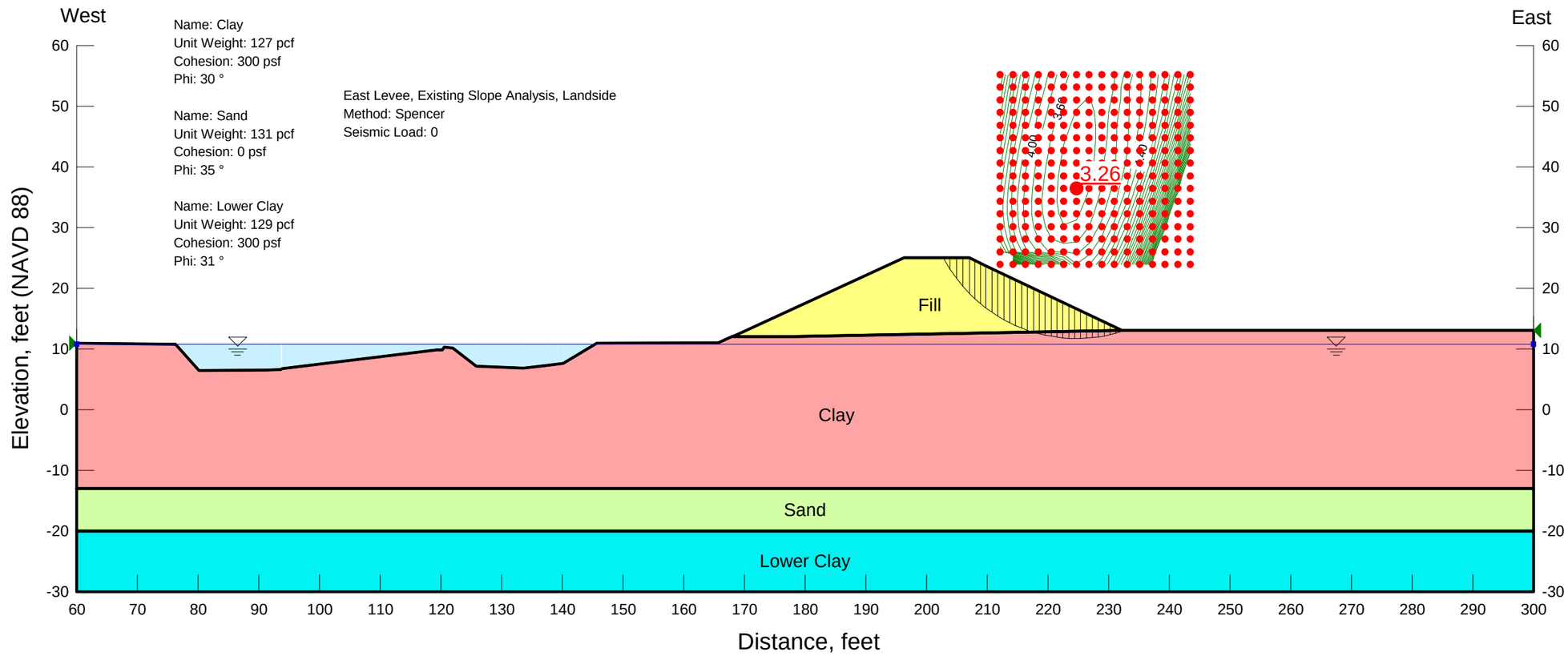
Name: Fill
Unit Weight: 120 pcf
Cohesion: 300 psf
Phi: 28 °

Name: Clay
Unit Weight: 127 pcf
Cohesion: 300 psf
Phi: 30 °

Name: Sand
Unit Weight: 131 pcf
Cohesion: 0 psf
Phi: 35 °

Name: Lower Clay
Unit Weight: 129 pcf
Cohesion: 300 psf
Phi: 31 °

East Levee, Existing Slope Analysis, Landside
Method: Spencer
Seismic Load: 0



amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Existing Condition East Levee, Reach B Levee Station 146+17	**Figure** 10-2
		Task 0B.03	Analyzed D. Levulett			
		Date 2/18/2009	Checked PSK			

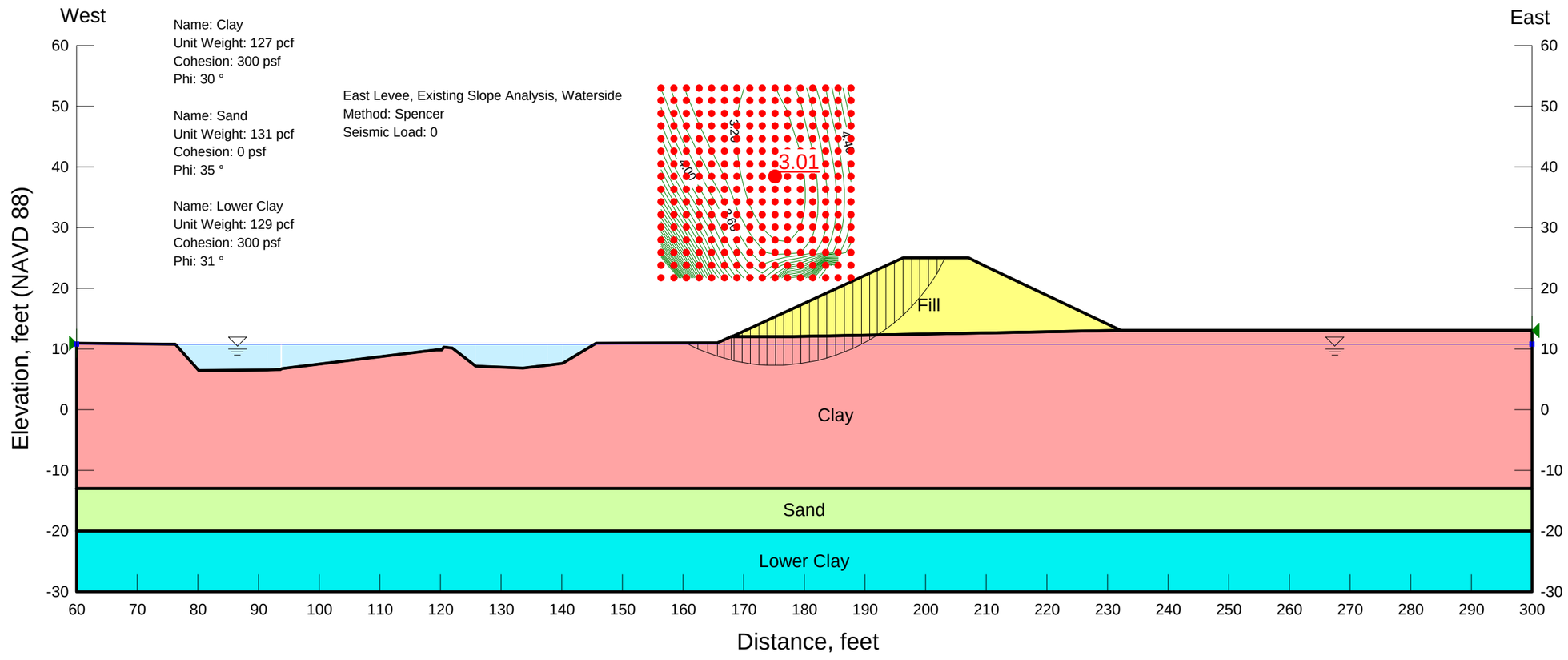
Name: Fill
Unit Weight: 120 pcf
Cohesion: 300 psf
Phi: 28 °

Name: Clay
Unit Weight: 127 pcf
Cohesion: 300 psf
Phi: 30 °

Name: Sand
Unit Weight: 131 pcf
Cohesion: 0 psf
Phi: 35 °

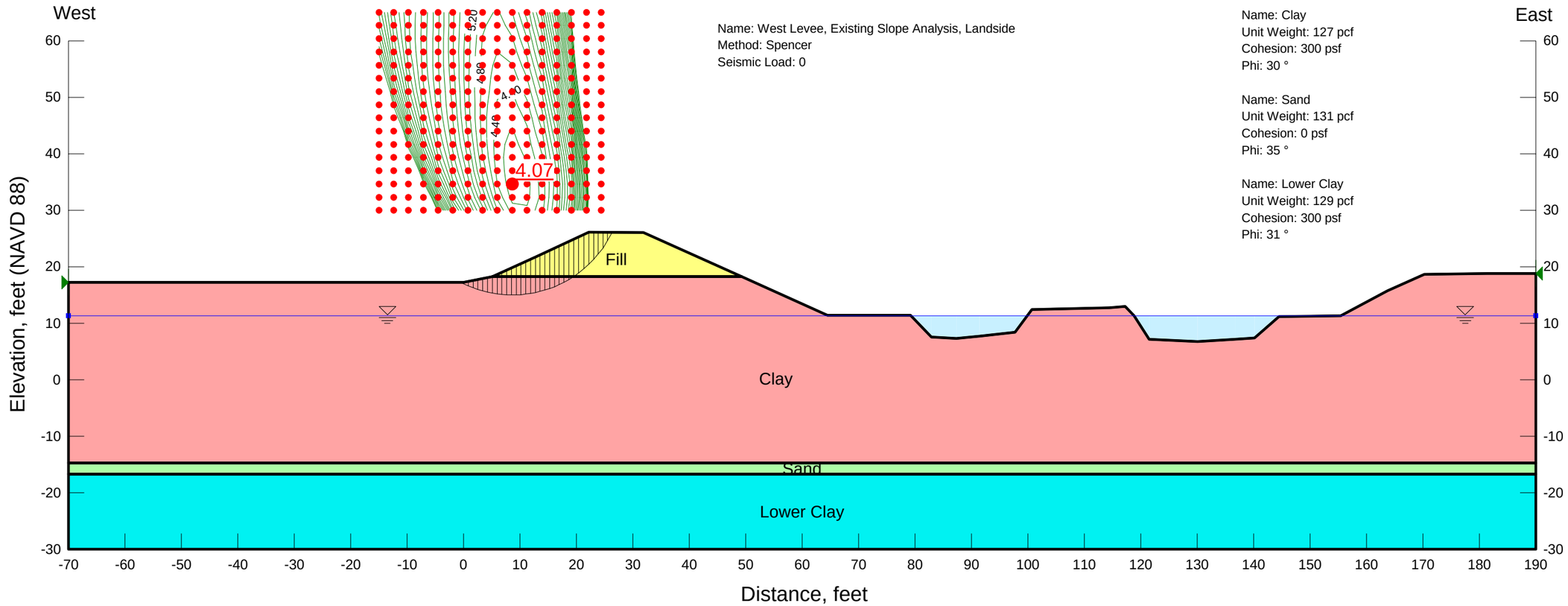
Name: Lower Clay
Unit Weight: 129 pcf
Cohesion: 300 psf
Phi: 31 °

East Levee, Existing Slope Analysis, Waterside
Method: Spencer
Seismic Load: 0



amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Existing Condition East Levee, Reach B Levee Station 146+17	**Figure** 10-3
		Task 0B.03	Analyzed D. Levulett			
		Date 2/18/2009	Checked PSK			

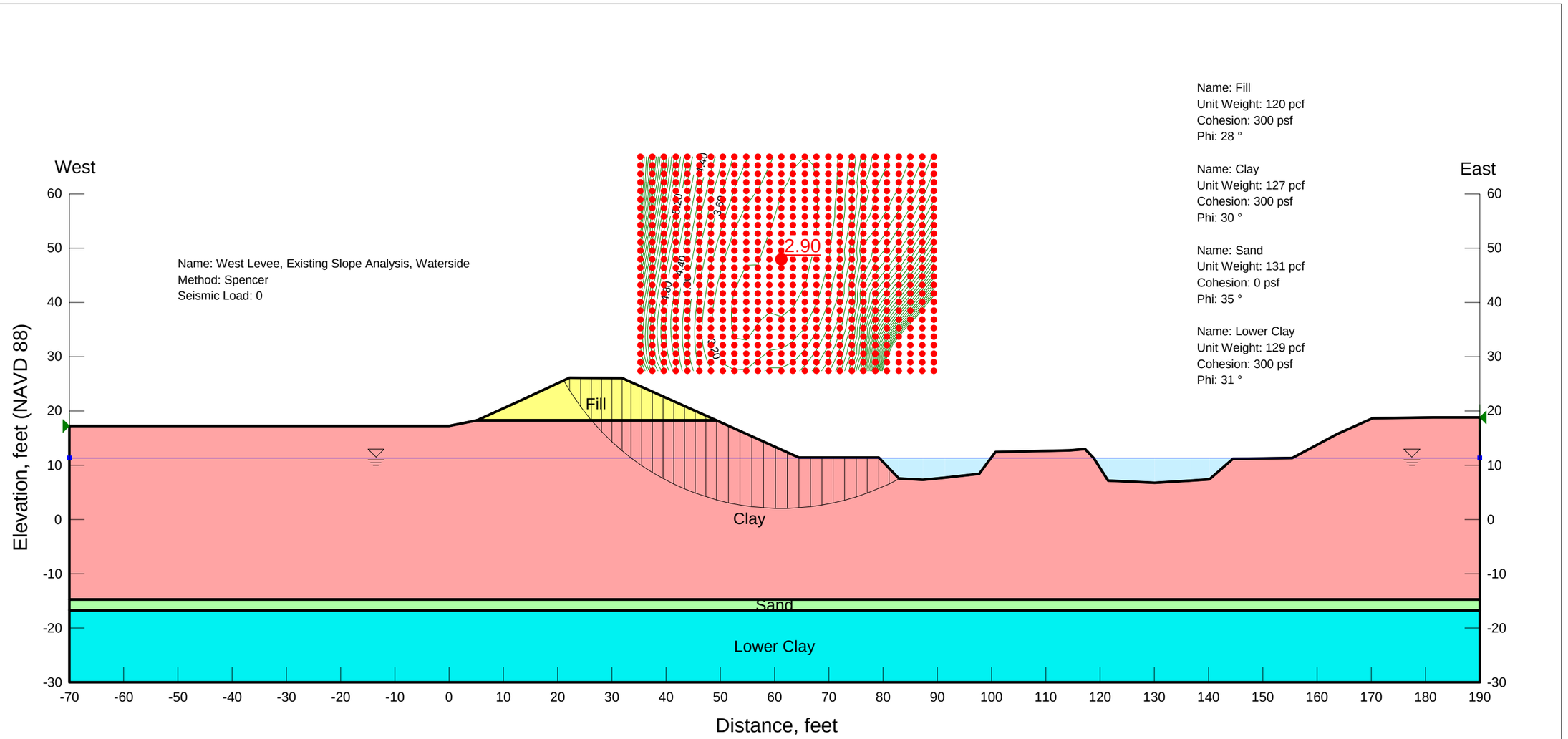
Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach B\East_B_RS_10502_Existing.gsz Date: 2/18/2009 Time: 11:42:18 AM Last Edited By: Levulett, Darcie



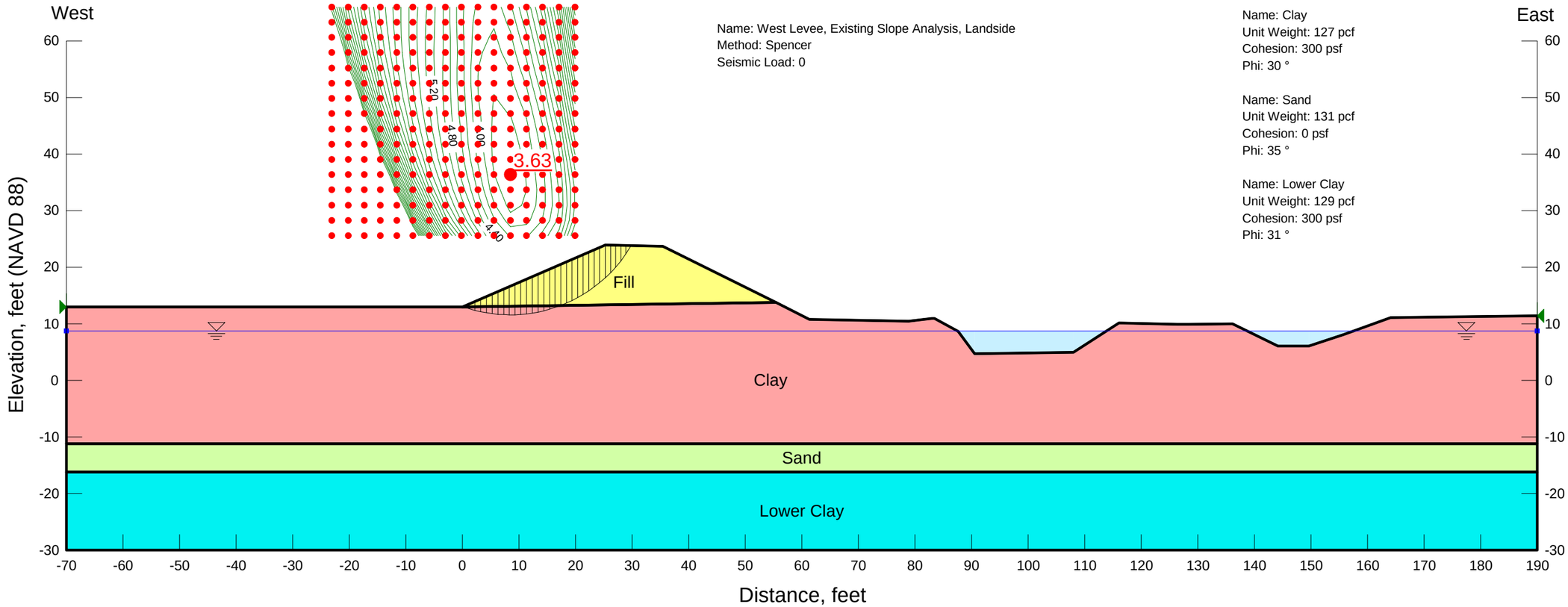
Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach CWest_C_RS_13923_Existing.gsz Date: 2/19/2009 Time: 9:41:15 AM Last Edited By: Levulett, Darcie

amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn S.Pulijala	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Existing Condition West Levee, Reach C Levee Station 139+23	**Figure** 10-4
		Task 0B.03	Analyzed S.Pulijala			
		Date 2/19/2009	Checked			

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach CWest_C_RS_13923_Existing.gsz Date: 2/17/2009 Time: 5:03:55 PM Last Edited By: Levulett, Darcie



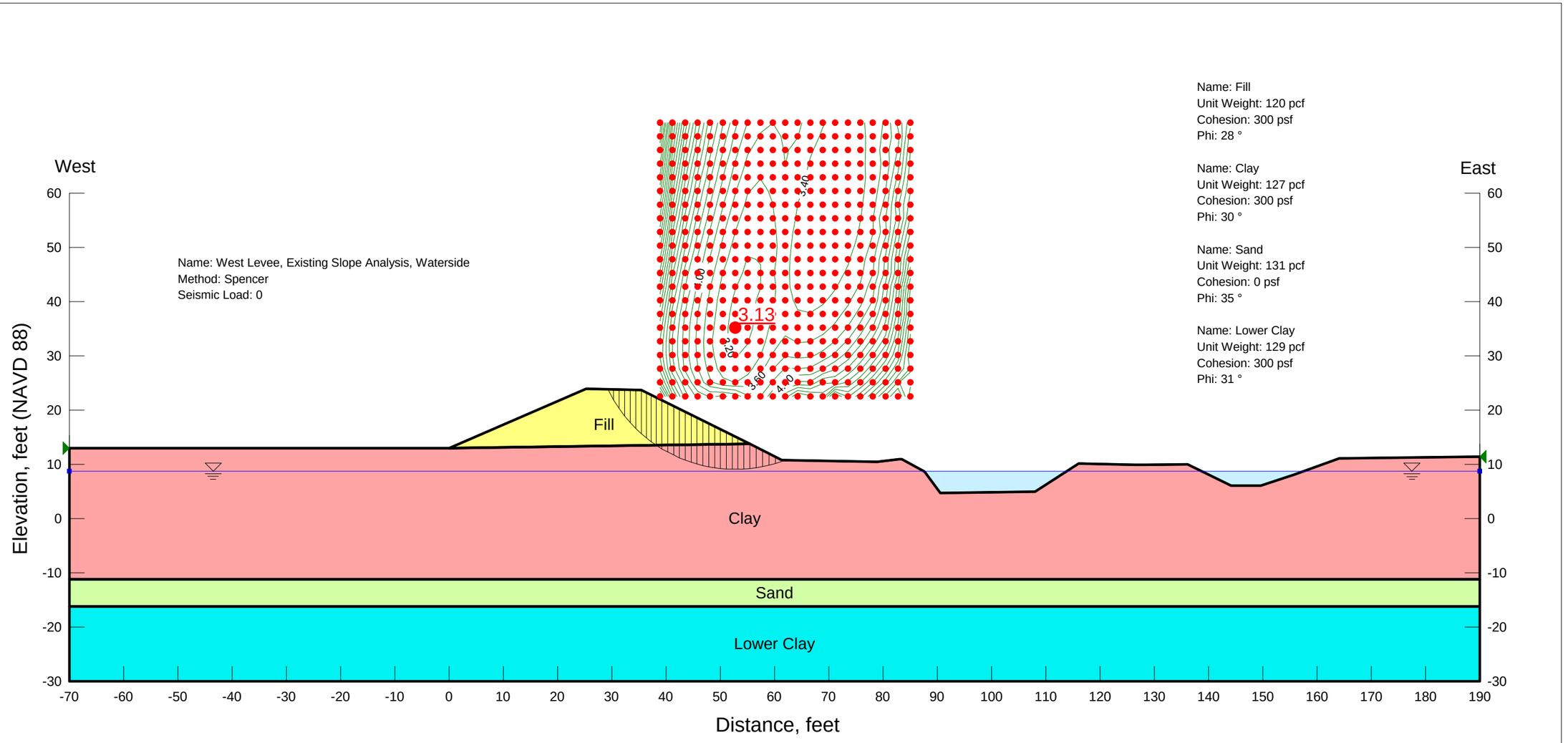
amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn S.Pulijala	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Existing Condition West Levee, Reach C Levee Station 139+23	**Figure** 10-5
		Task 0B.03	Analyzed S.Pulijala			
		Date 2/17/2009	Checked			



amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Existing Condition West Levee, Reach D Levee Station 159+18	**Figure** 10-6
		Task 0B.03	Analyzed D. Levulett			
		Date 2/17/2009	Checked psk			

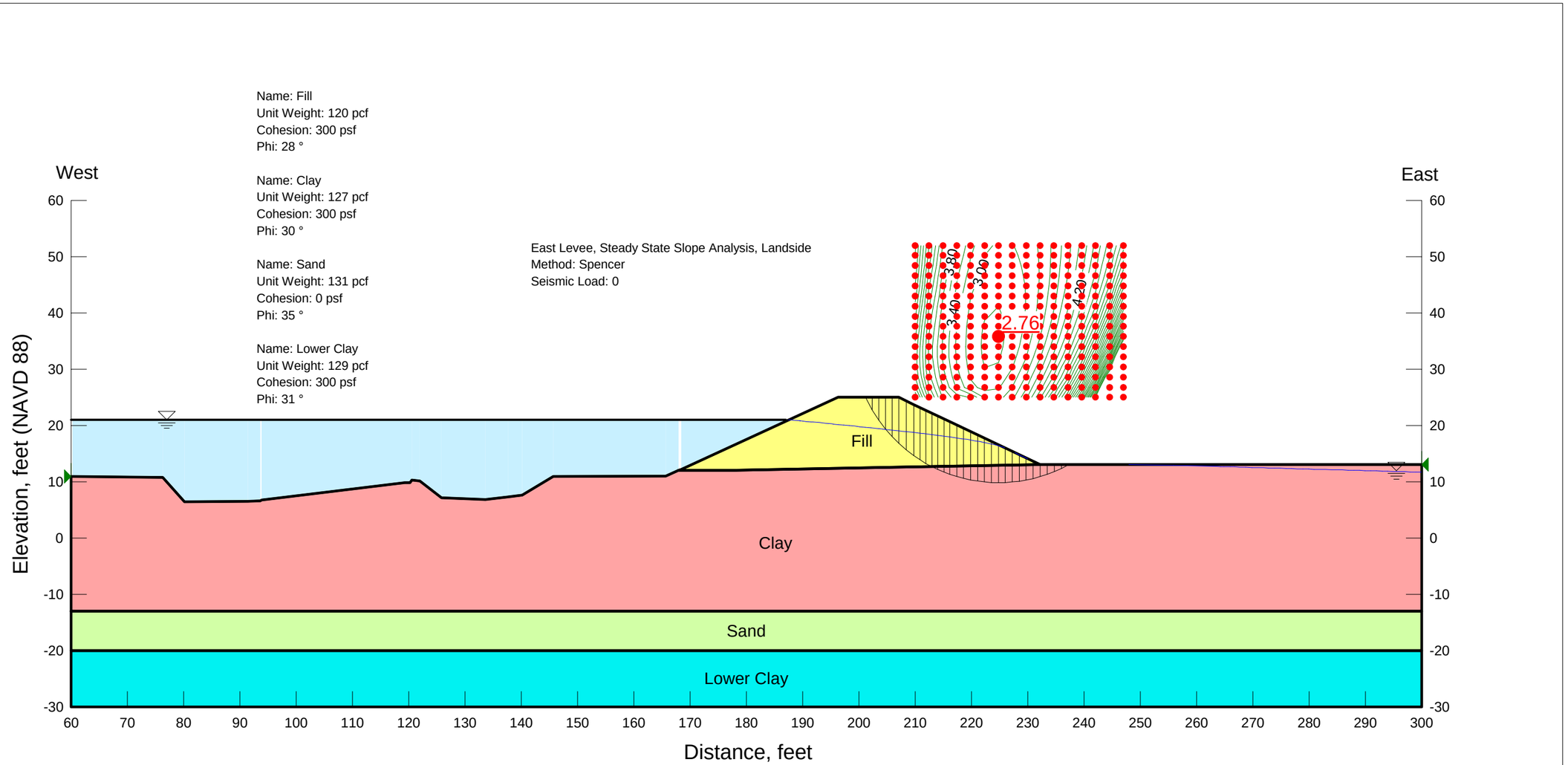
Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach D\West_D_RS_9501_Existing.gsz Date: 2/17/2009 Time: 5:02:46 PM Last Edited By: Levulett, Darcie

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach D\West_D_RS_9501_Existing.gsz Date: 2/17/2009 Time: 5:02:46 PM Last Edited By: Levulett, Darcie



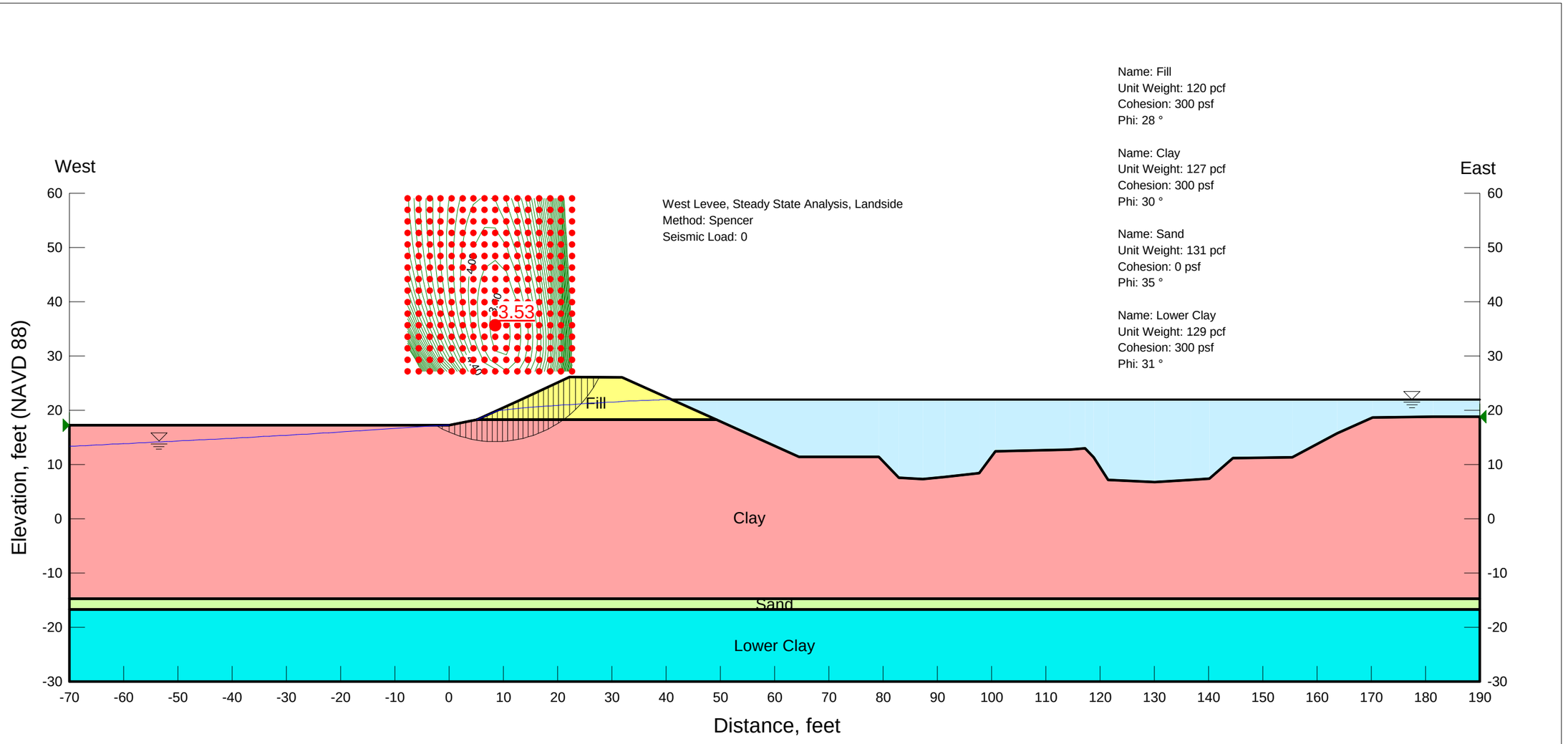
amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Existing Condition West Levee, Reach D Levee Station 159+18	**Figure** 10-7
		Task 0B.03	Analyzed D. Levulett			
		Date 2/17/2009	Checked psk			

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach B\East_B_RS_10502_SteadyState.gsz Date: 2/27/2009 Time: 1:57:23 PM Last Edited By: Levulett, Darcie



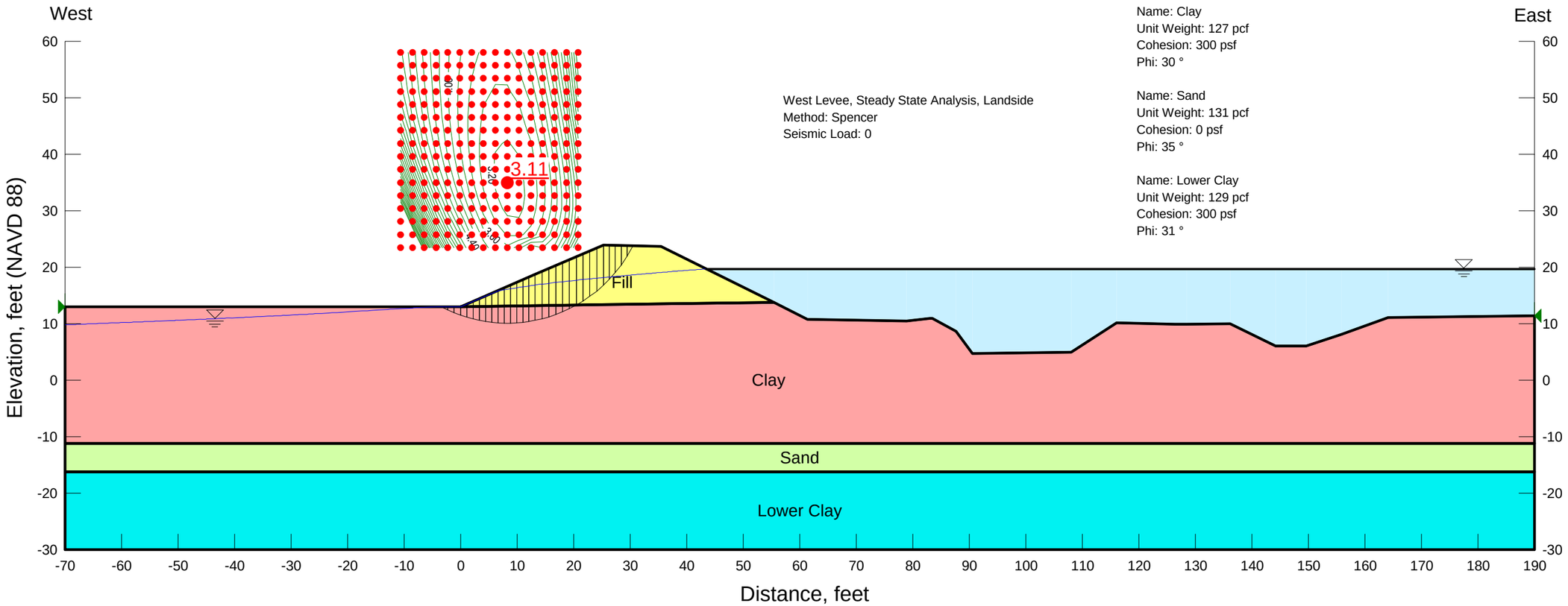
amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Steady State Condition East Levee, Reach B Station 146+17	**Figure** 11-1
		Task 0B.03	Analyzed D. Levulett			
		Date 2/27/2009	Checked PSK			

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach C\West_C_RS_13923_SteadyState.gsz Date: 2/27/2009 Time: 2:04:08 PM Last Edited By: Levulett, Darcie



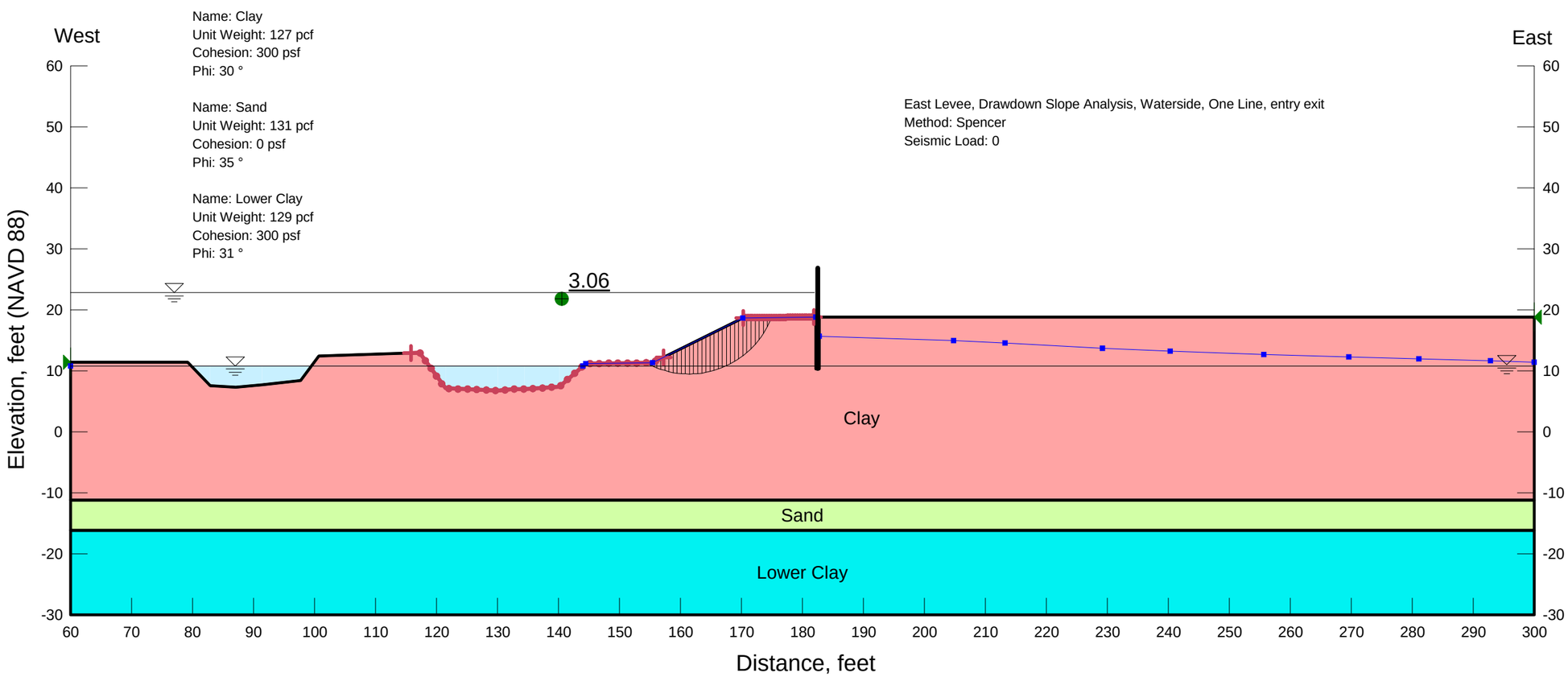
amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Steady State Condition West Levee, Reach C Levee Station 139+23	**Figure** 11-2
		Task 0B.03	Analyzed D. Levulett			
		Date 2/27/2009	Checked PSK			

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach DWest_D_RS_9501_SteadyState.gsz Date: 4/29/2009 Time: 1:19:12 PM Last Edited By: Levulett, Darcie



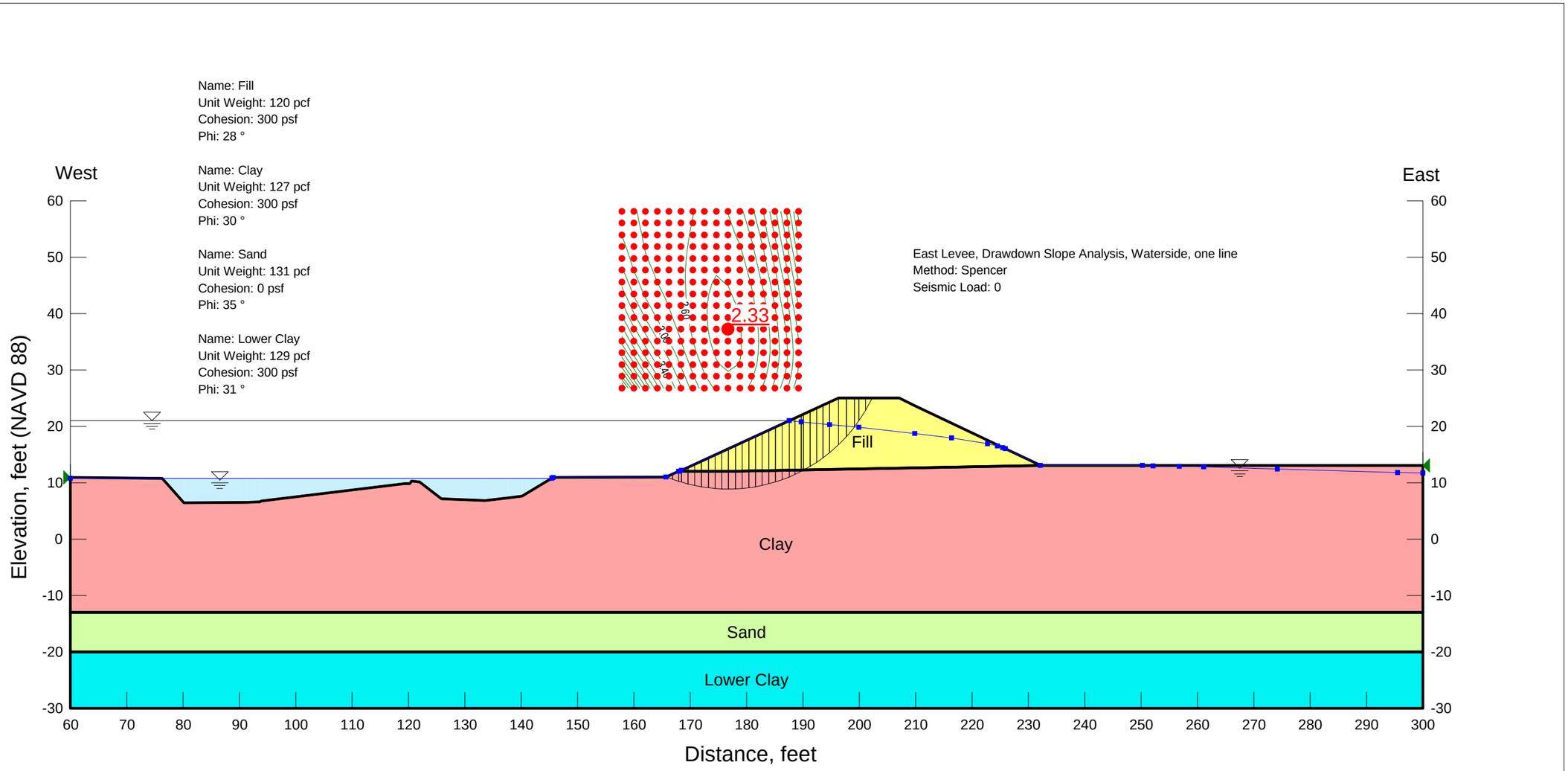
amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Steady State Condidtion West Levee, Reach D Levee Station 159+18	**Figure** 11-3
		Task 0B.03	Analyzed D. Levulett			
		Date 4/29/2009	Checked PSK			

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach A\East_A_RS_11496_Drawdown.gsz Date: 3/2/2009 Time: 10:43:43 AM Last Edited By: Levulett, Darcie



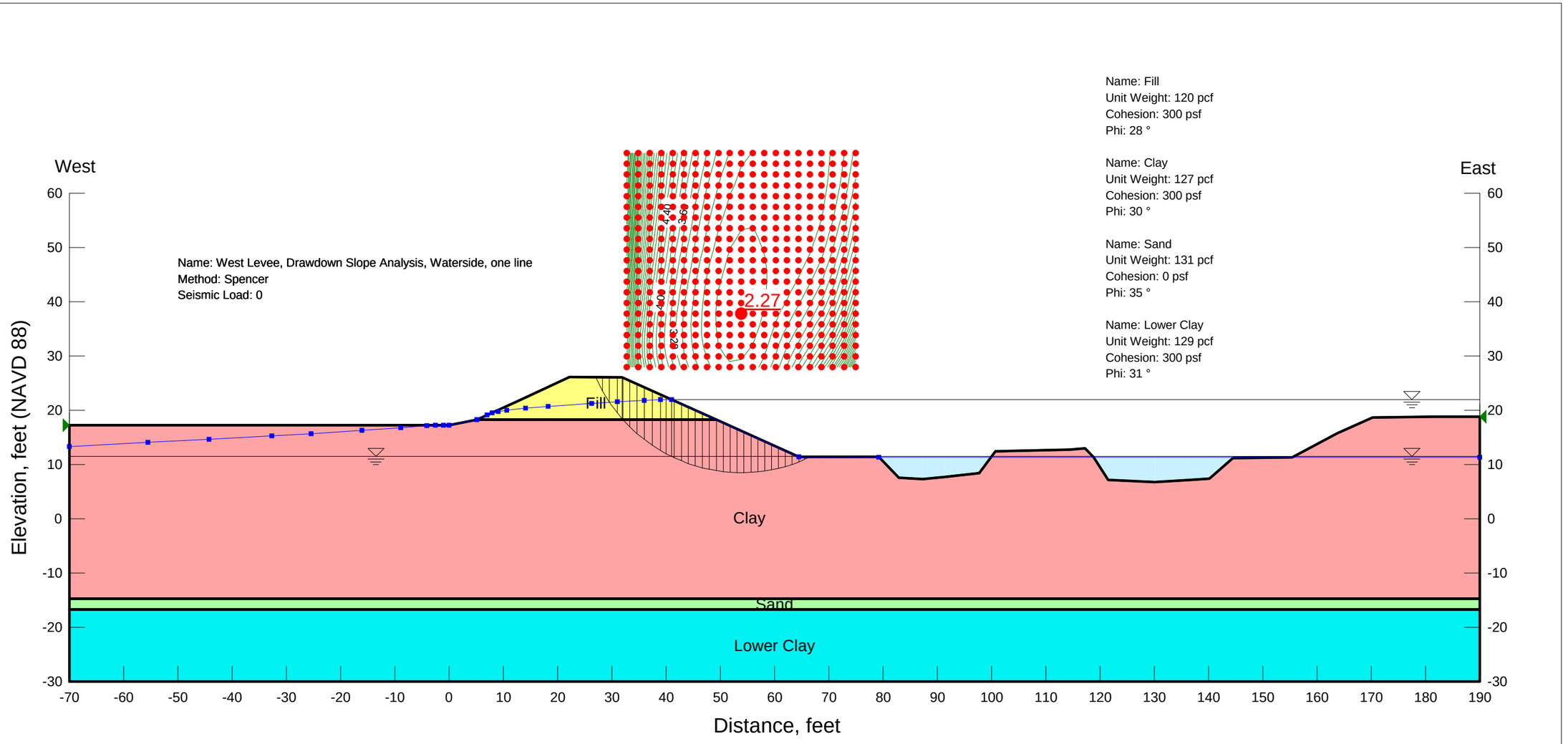
amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Drawdown Condition East Levee, Reach A Levee Station 139+23	**Figure** 12-1
		Task 0B.03	Analyzed S. Pulijala			
		Date 3/2/2009	Checked			

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach B\East_B_RS_10502_Drawdown.gsz Date: 2/27/2009 Time: 9:40:09 AM Last Edited By: Levulett, Darcie



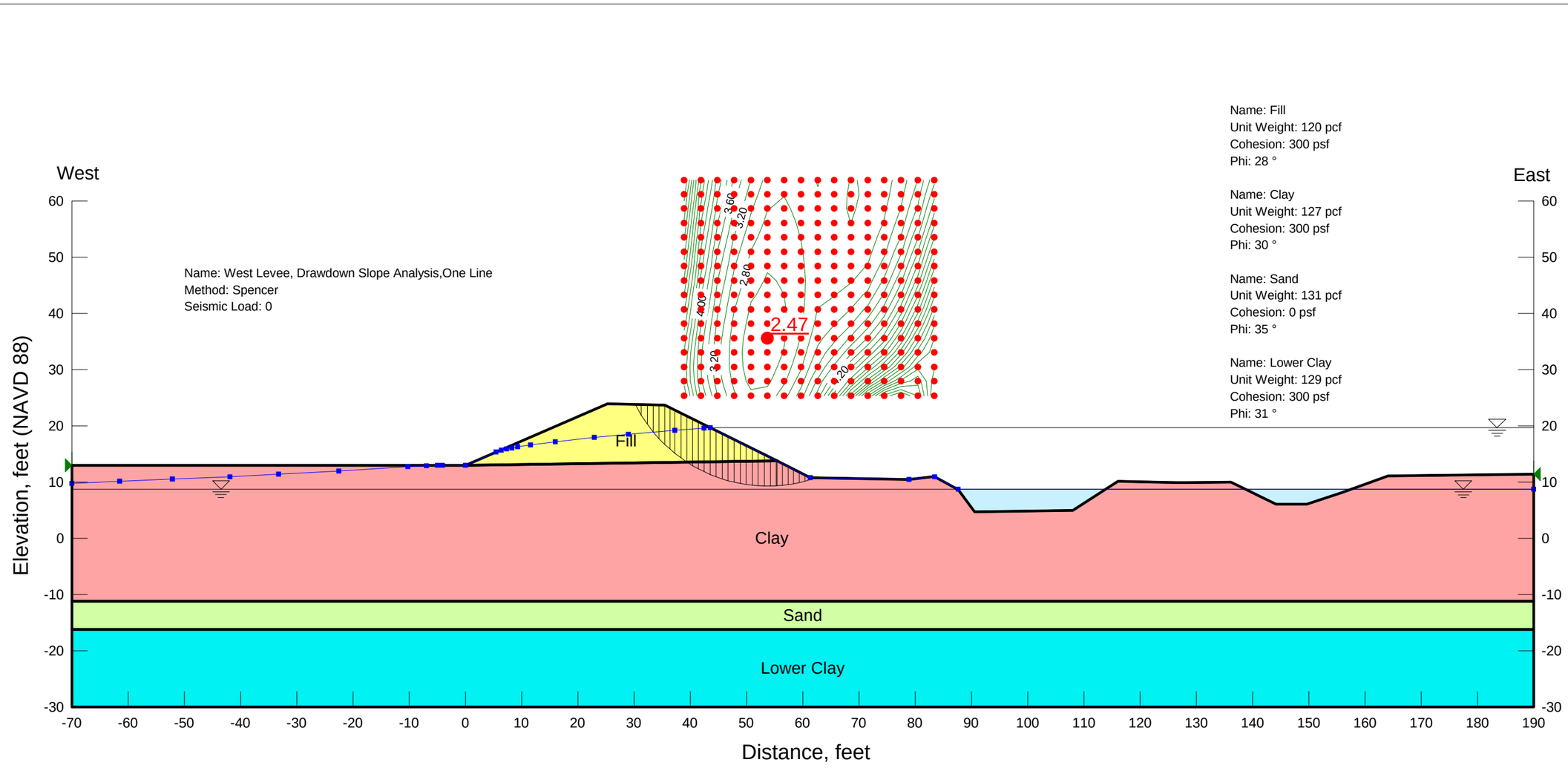
amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Drawdown Condition East Levee, Reach B Levee Station 146+17	**Figure** 12-2
		Task 0B.03	Analyzed D. Levulett			
		Date 2/27/2009	Checked PSK			

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach C\West_C_RS_13923_Drawdown.gsz Date: 3/2/2009 Time: 10:58:59 AM Last Edited By: Levulett, Darcie



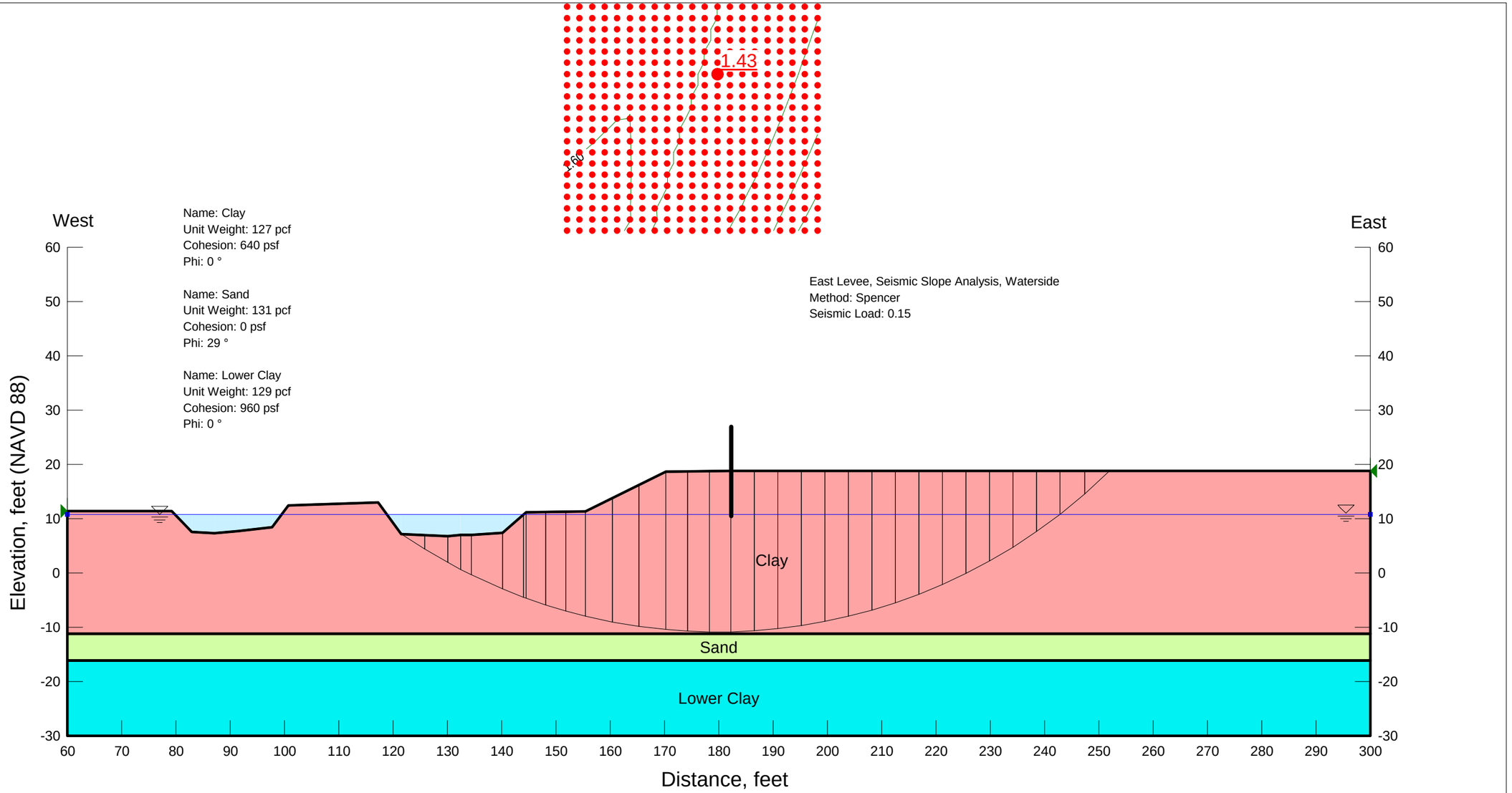
amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Drawdown Condition West Levee, Reach C Levee Station 139+23	**Figure** 12-3
		Task 0B.03	Analyzed S. Pulijala			
		Date 3/2/2009	Checked			

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach D\West_D_RS_9501_Drawdown.gsz Date: 2/27/2009 Time: 1:56:43 PM Last Edited By: Pulijala, Sreekar



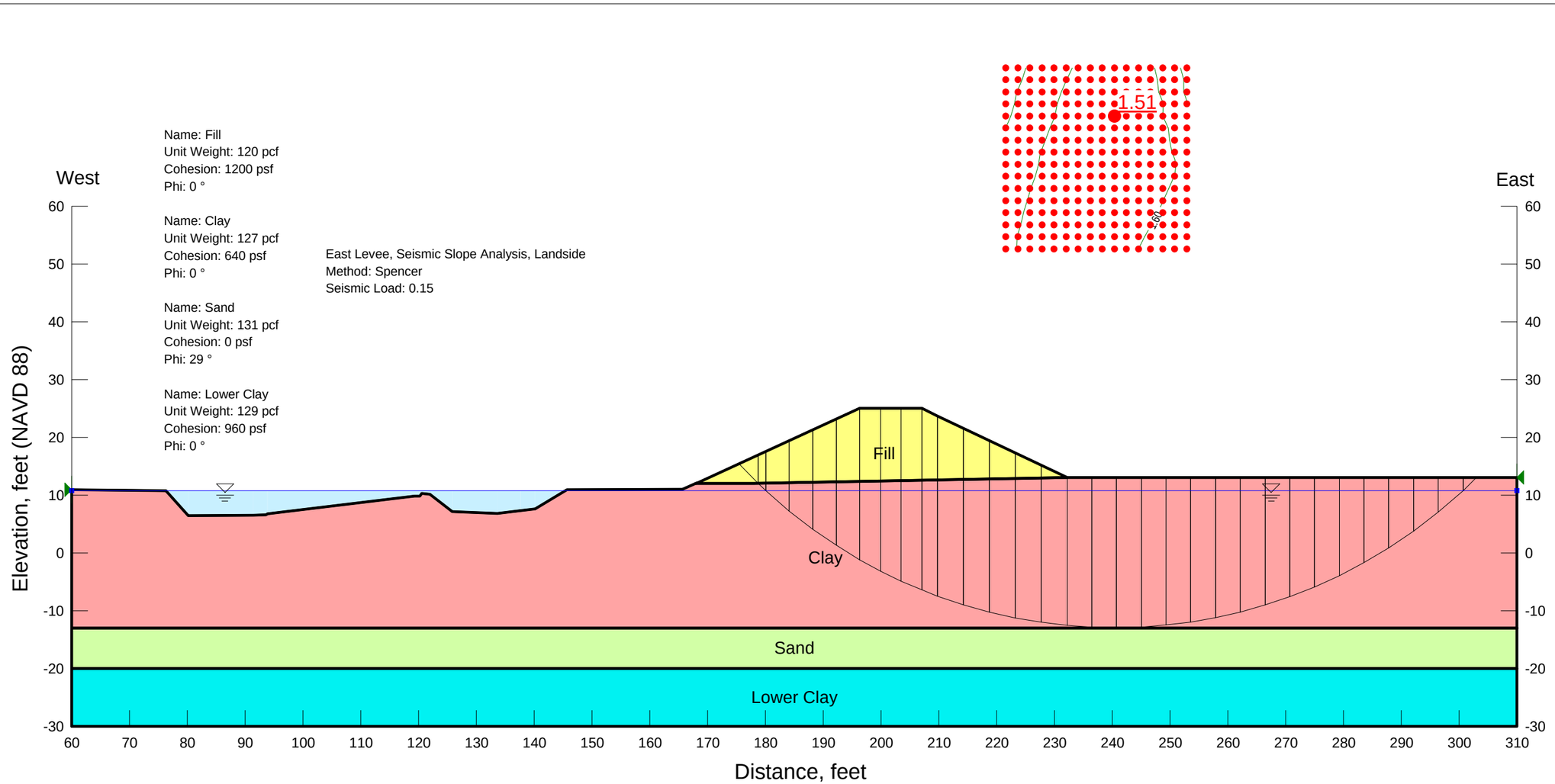
amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Drawdown Condition West Levee, Reach D Levee Station 159+18	**Figure** 12-4
		Task 0B.03	Analyzed S. Pulijala			
		Date 2/27/2009	Checked			

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach A\East_A_RS_11496_Seismic.gsz Date: 5/1/2009 Time: 10:47:10 AM Last Edited By: Levulett, Darcie



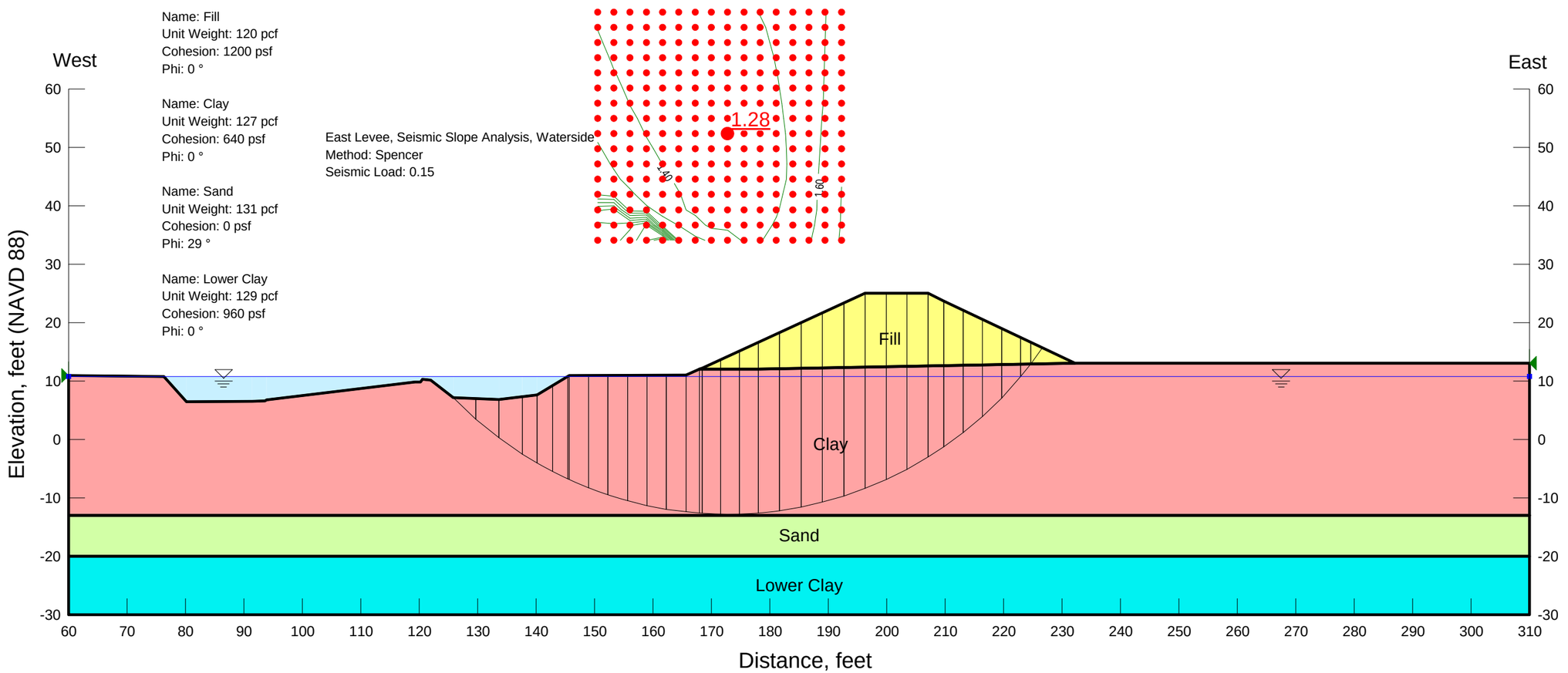
amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Seismic Condition East Levee, Reach A Levee Station 139+23	**Figure** 13-1
		Task 0B.03	Analyzed D. Levulett			
		Date 5/1/2009	Checked PSK			

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach B\East_B_RS_10502_Seismic.gsz Date: 5/1/2009 Time: 10:54:08 AM Last Edited By: Levulett, Darcie

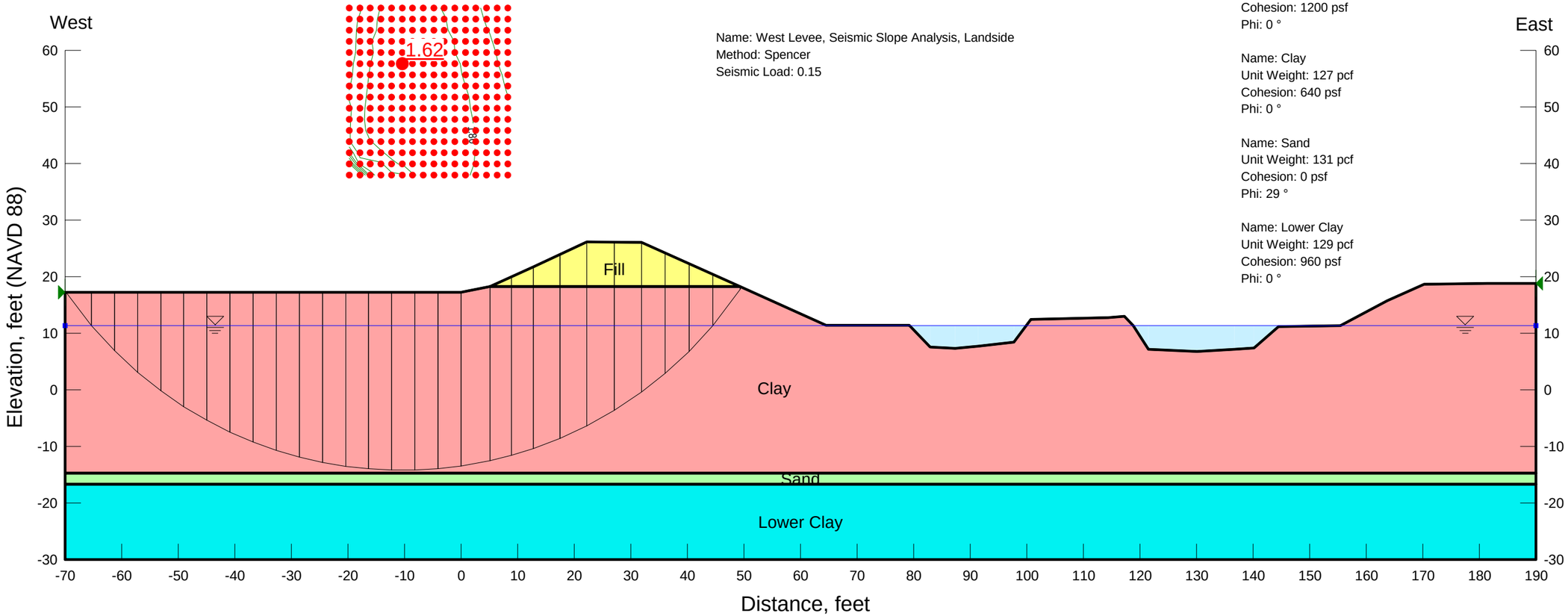


amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Seismic Condition East Levee, Reach B Levee Station 146+17	**Figure** 13-2
		Task 0B.03	Analyzed D. Levulett			
		Date 5/1/2009	Checked PSK			

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach B\East_B_RS_10502_Seismic.gsz Date: 5/1/2009 Time: 10:54:08 AM Last Edited By: Levulett, Darcie

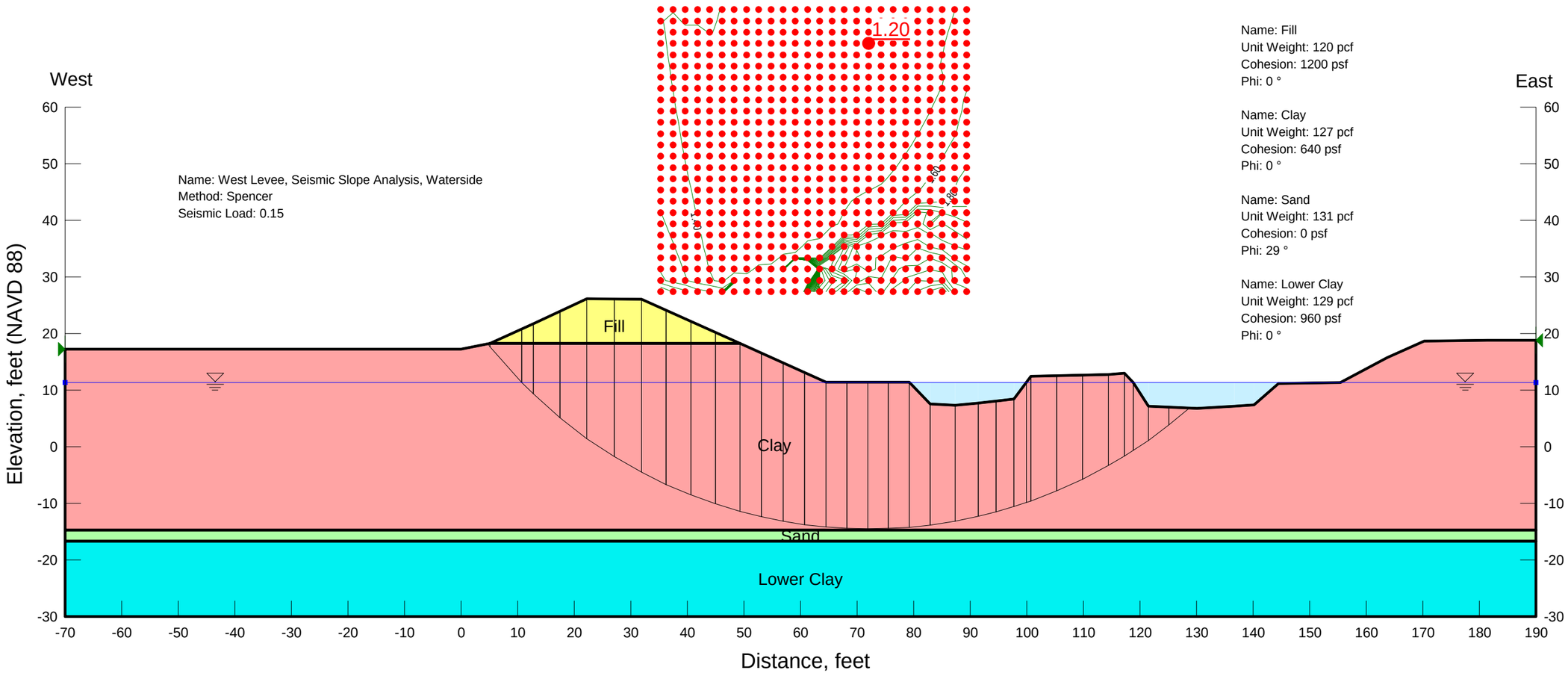


amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Seismic Condition East Levee, Reach B Levee Station 146+17	**Figure** 13-3
		Task 0B.03	Analyzed D. Levulett			
		Date 5/1/2009	Checked PSK			



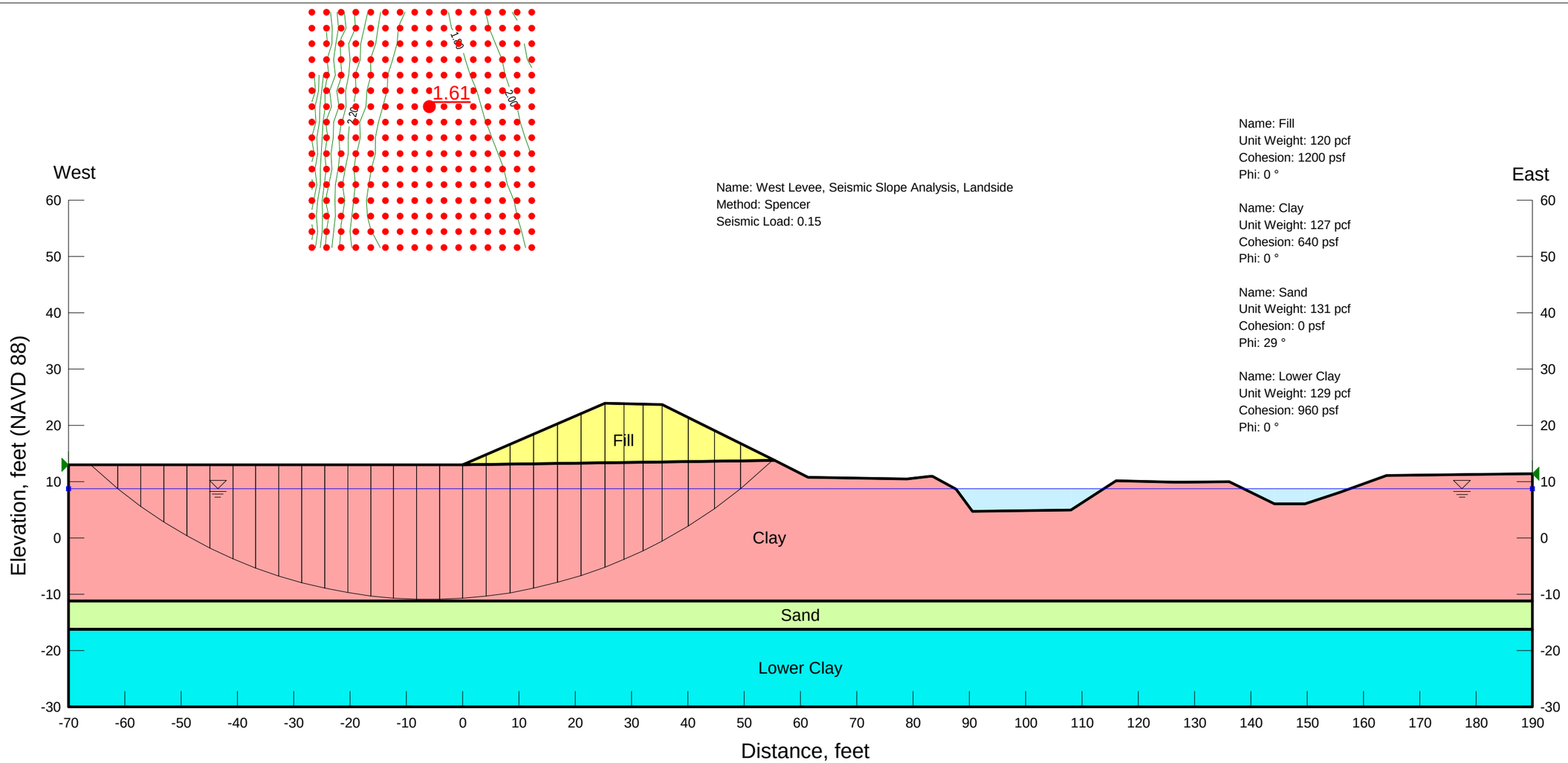
Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach C\West_C_RS_13923_Seismic.gsz Date: 5/1/2009 Time: 10:57:55 AM Last Edited By: Levulett, Darcie

amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Seismic Condition West Levee, Reach C Levee Station 139+23	**Figure** 13-4
		Task 0B.03	Analyzed D. Levulett			
		Date 5/1/2009	Checked PSK			



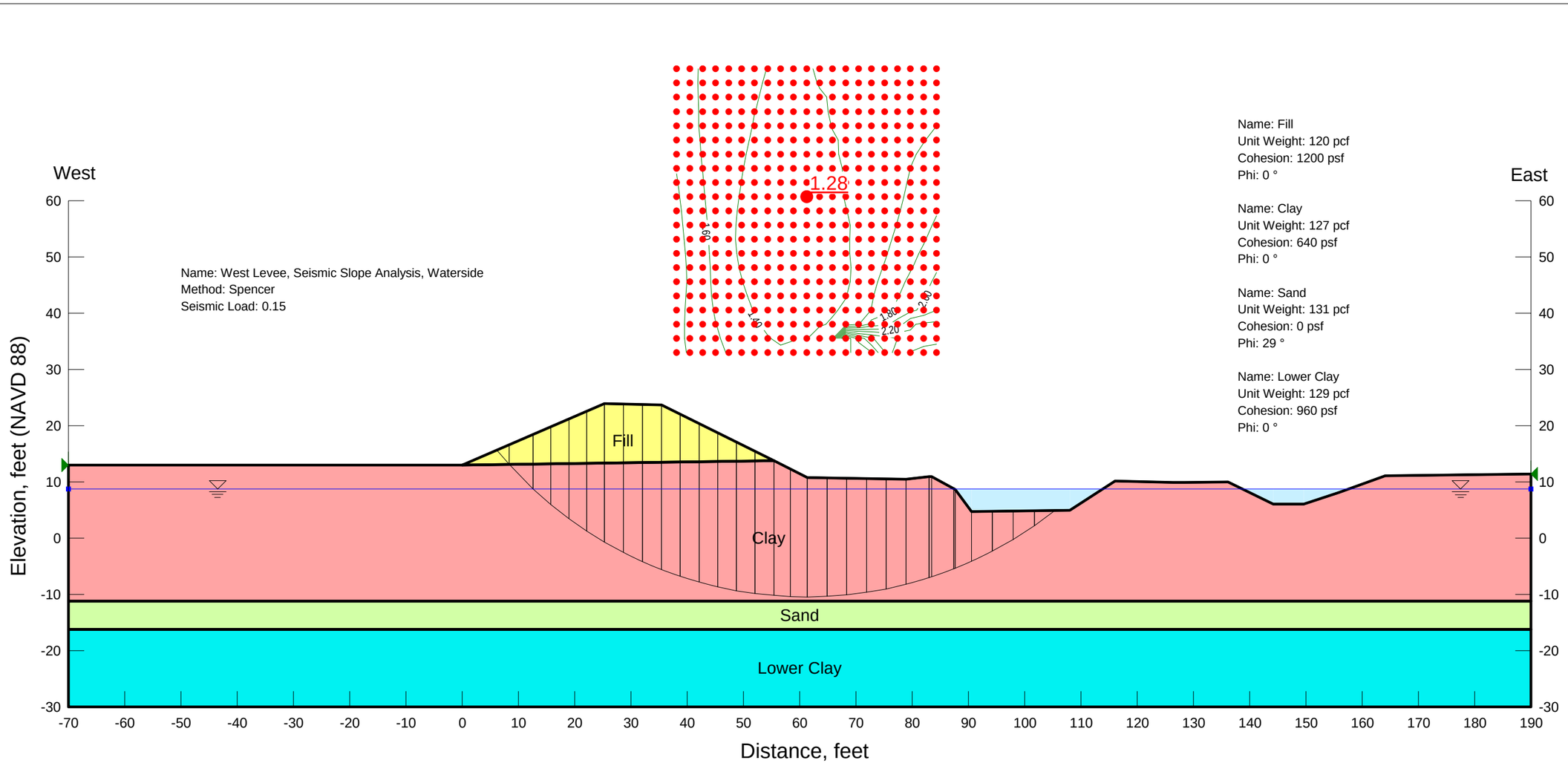
amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Seismic Condition West Levee, Reach C Levee Station 139+23	**Figure** 13-5
		Task 0B.03	Analyzed D. Levulett			
		Date 5/1/2009	Checked PSK			

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\GeoStudio\Reach CWest_C_RS_13923_Seismic.gsz Date: 5/1/2009 Time: 10:57:55 AM Last Edited By: Levulett, Darcie



Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach DWest_D_RS_9501_Seismic.gsz Date: 5/1/2009 Time: 11:05:28 AM Last Edited By: Levulett, Darcie

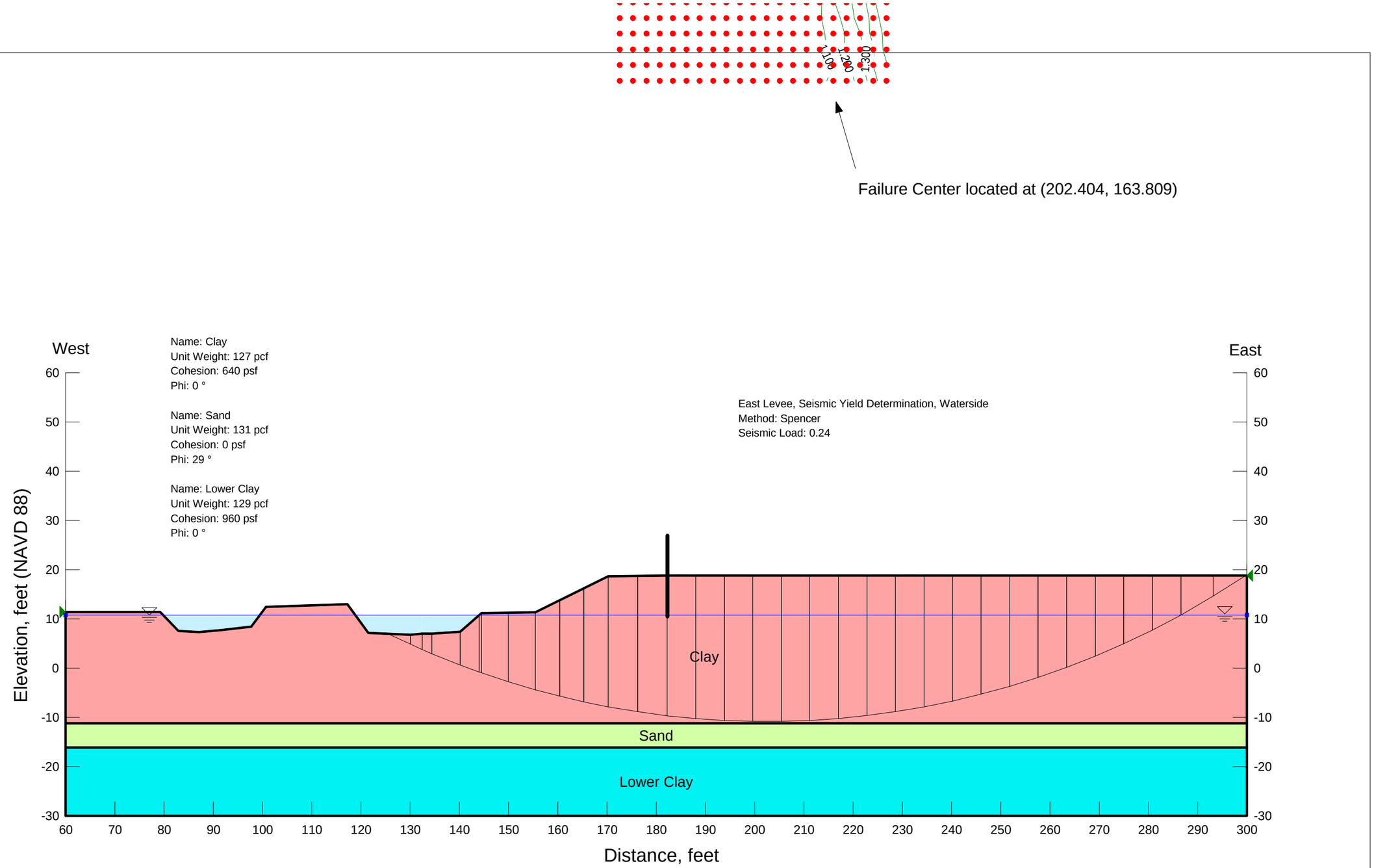
amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Seismic Condition West Levee, Reach D Levee Station 159+18	**Figure** 13-6
		Task 0B.03	Analyzed D. Levulett			
		Date 5/1/2009	Checked PSK			



amec	**Scale** 1 inch = 20 feet	Project Number 13815.002 Task 0B.03 Date 5/1/2009	Drawn D. Levulett Analyzed D. Levulett Checked PSK	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Seismic Condition West Levee, Reach D Levee Station 159+18	**Figure** 13-7

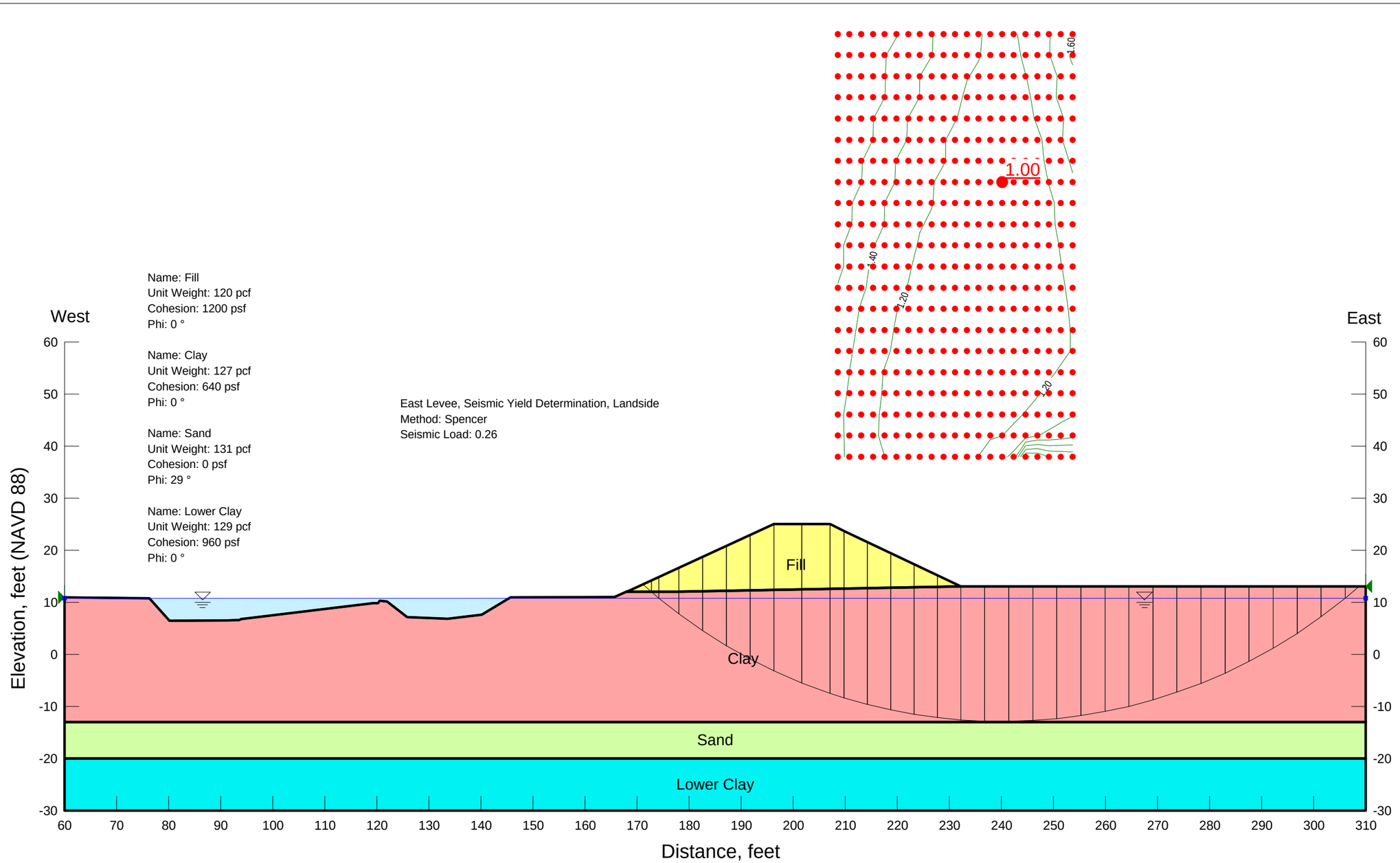
Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach DWest_D_RS_9501_Seismic.gsz Date: 5/1/2009 Time: 11:05:28 AM Last Edited By: Levulett, Darcie

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach A\East_A_RS_11496_SeismicK.gsz Date: 5/1/2009 Time: 10:50:57 AM Last Edited By: Levulett, Darcie



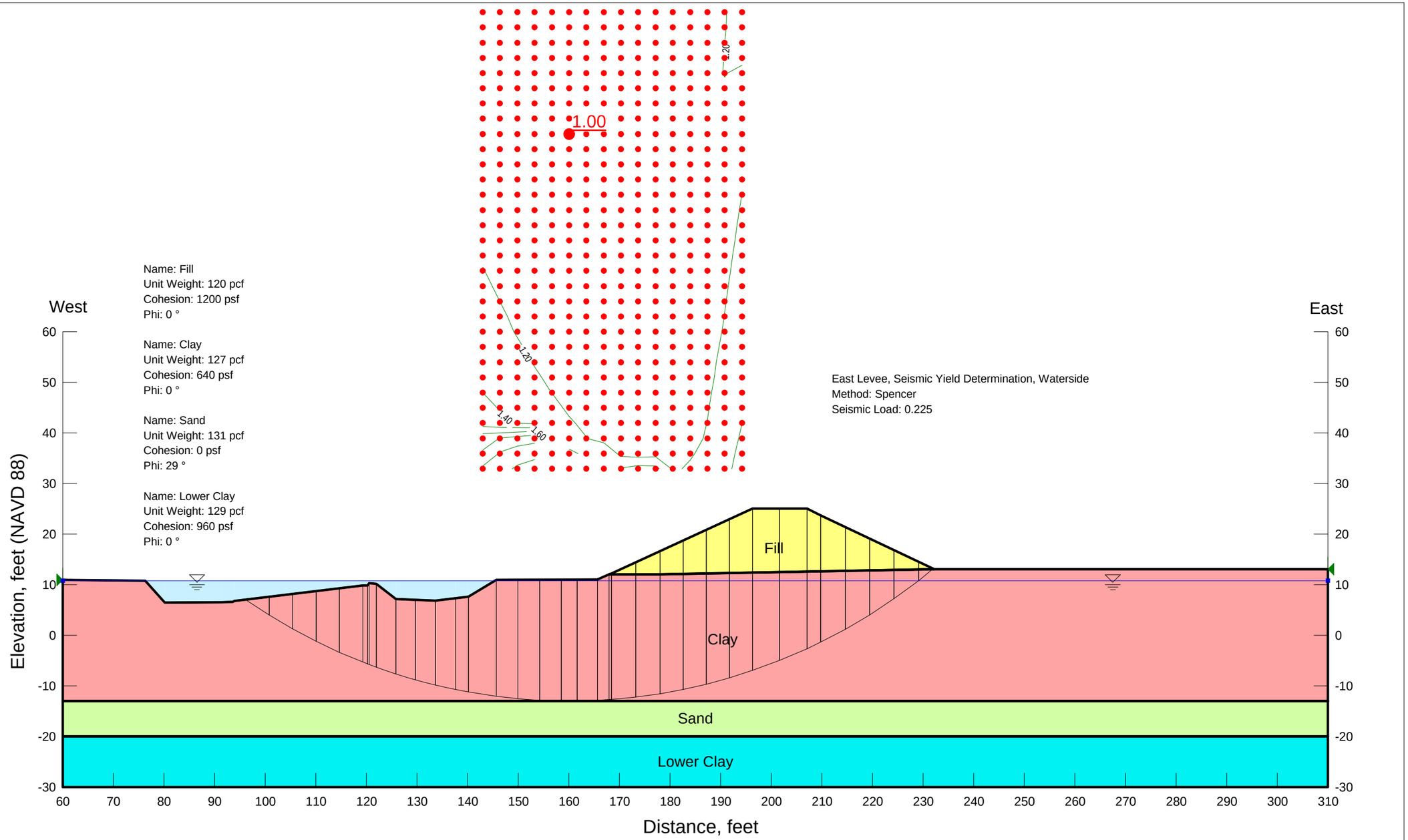
amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Seismic Yield Determination East Levee, Reach A Levee Station 139+23	**Figure** 14-1
		Task 0B.03	Analyzed D. Levulett			
		Date 5/1/2009	Checked PSK			

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach B\East_B_RS_10502_SeismicK.gsz Date: 5/1/2009 Time: 1:27:06 PM Last Edited By: Levulett, Darcie



amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Seismic Yield Determination East Levee, Reach B Levee Station 146+17
		Task 0B.03	Analyzed D. Levulett		**Figure** 14-2
		Date 5/1/2009	Checked PSK		

Directory: I:\Project\13000013815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach B\East_B_RS_10502_SeismicK.gsz Date: 5/1/2009 Time: 1:27:06 PM Last Edited By: Levulett, Darcie



Name: Fill
Unit Weight: 120 pcf
Cohesion: 1200 psf
Phi: 0 °

Name: Clay
Unit Weight: 127 pcf
Cohesion: 640 psf
Phi: 0 °

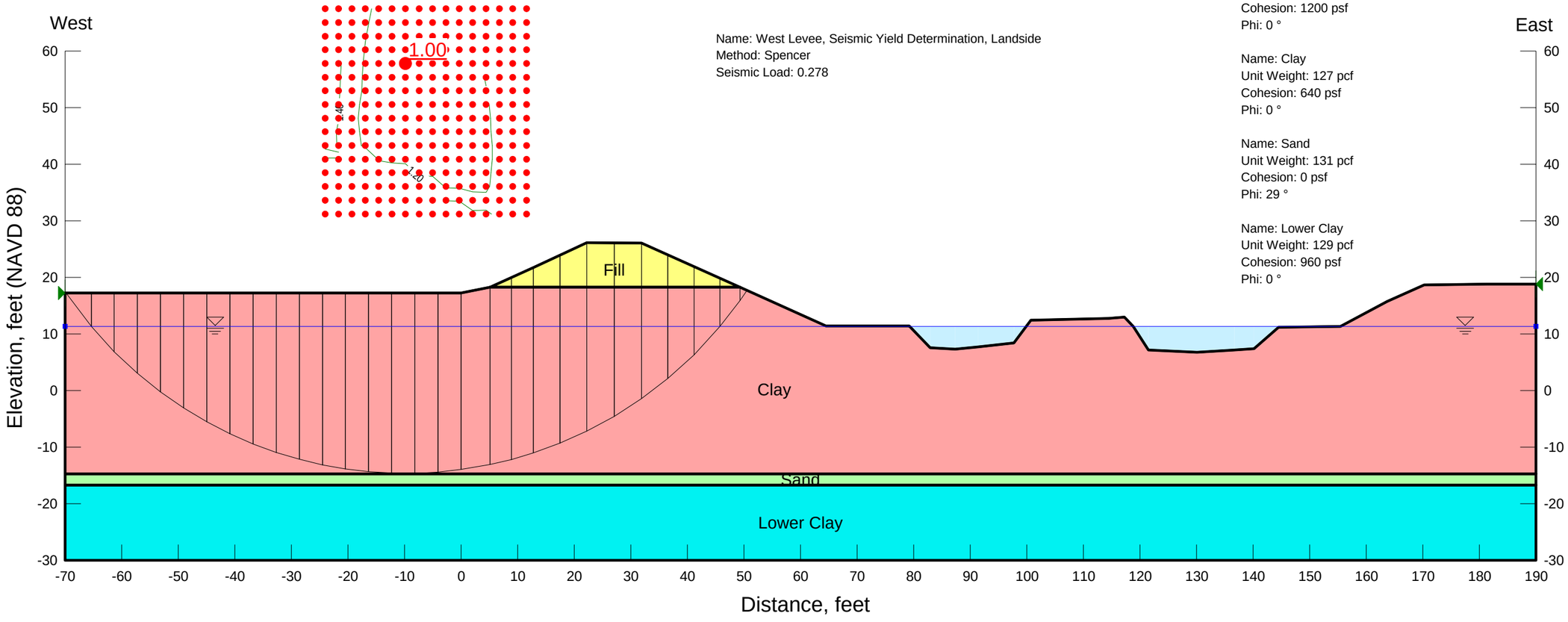
Name: Sand
Unit Weight: 131 pcf
Cohesion: 0 psf
Phi: 29 °

Name: Lower Clay
Unit Weight: 129 pcf
Cohesion: 960 psf
Phi: 0 °

East Levee, Seismic Yield Determination, Waterside
Method: Spencer
Seismic Load: 0.225

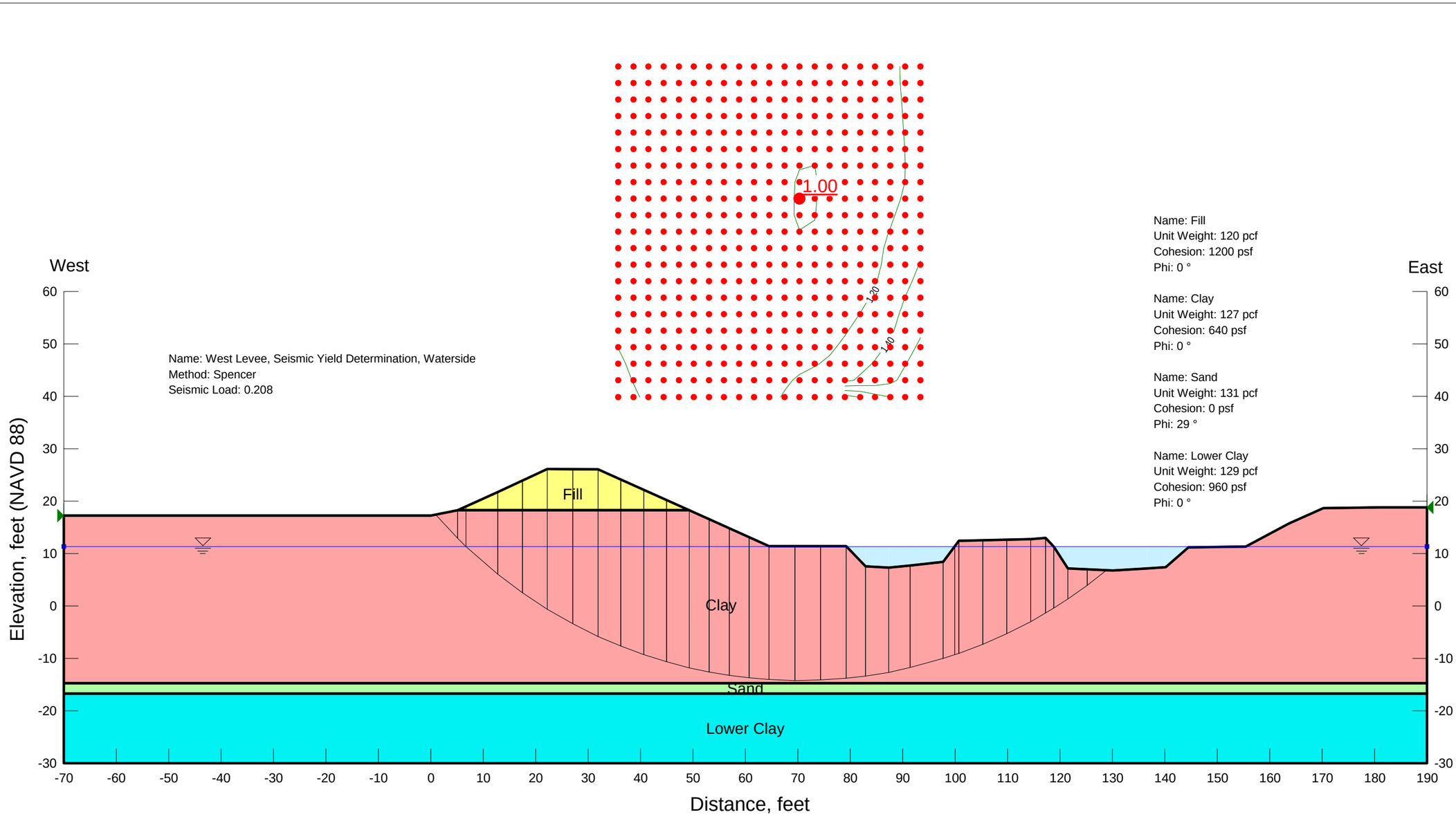
amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Seismic Yield Determination East Levee, Reach B Levee Station 146+17	**Figure** 14-3
		Task 0B.03	Analyzed D. Levulett			
		Date 5/1/2009	Checked PSK			

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach C\West_C_RS_13923_SeismicK.gsz Date: 5/1/2009 Time: 11:01:19 AM Last Edited By: Levulett, Darcie



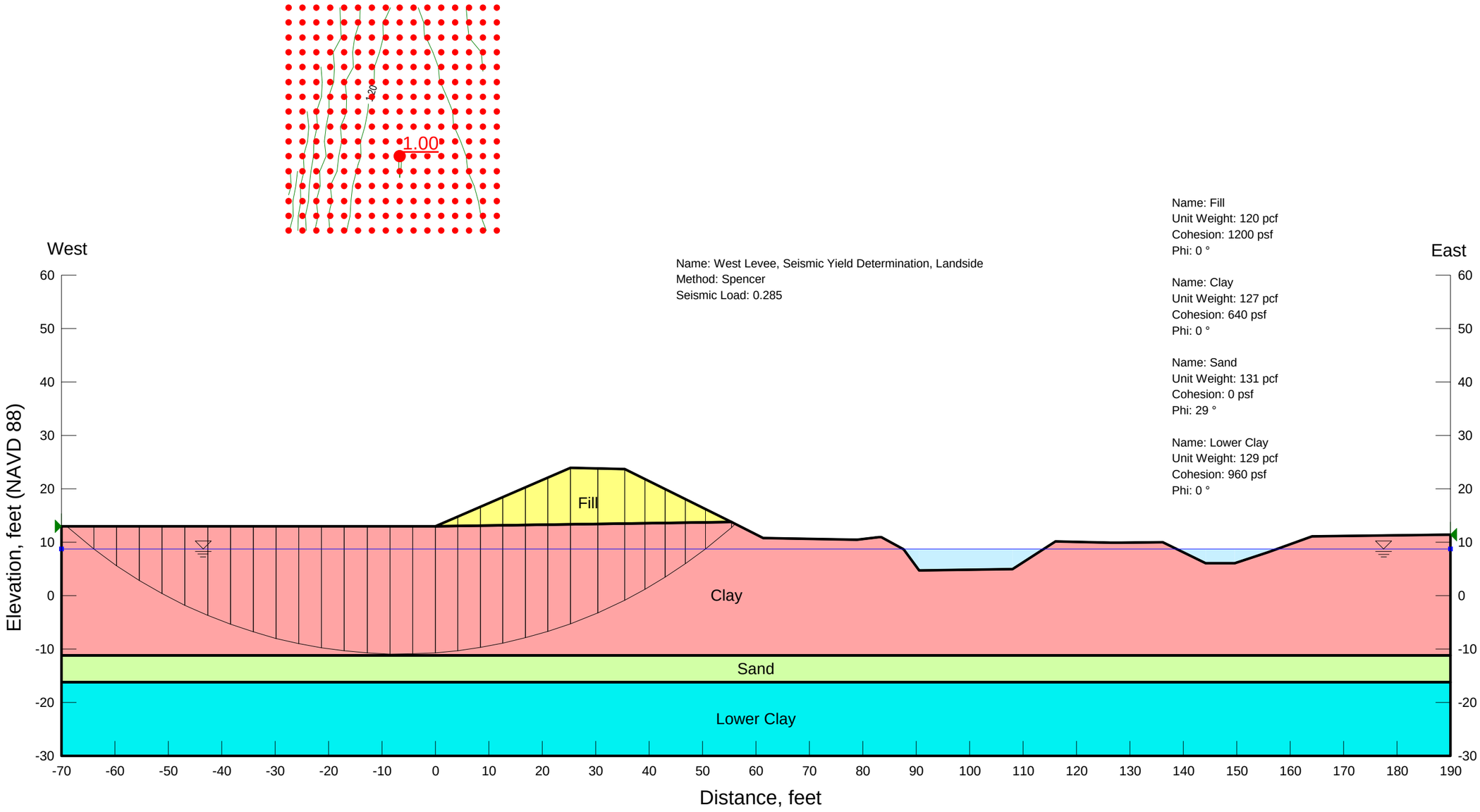
amec	**Scale** 1 inch = 20 feet	Project Number 13815.002 Task 0B.03 Date 5/1/2009	Drawn D. Levulett Analyzed D. Levulett Checked PSK	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Seismic Yield Determination West Levee, Reach C Levee Station 139+23	**Figure** 14-4

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach C\West_C_RS_13923_SeismicK.gsz Date: 5/1/2009 Time: 11:01:19 AM Last Edited By: Levulett, Darcie



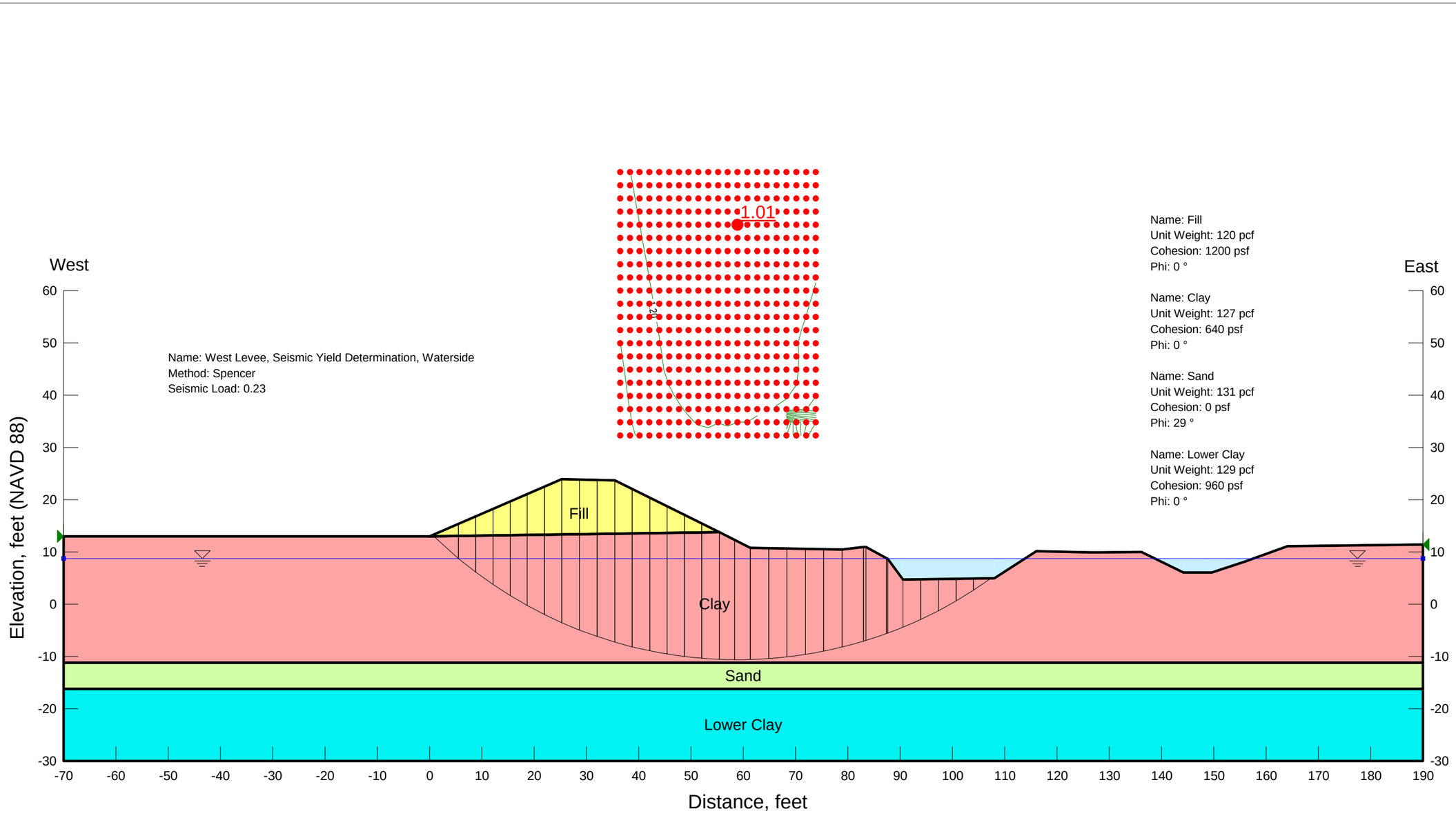
amec	**Scale** 1 inch = 20 feet	Project Number 13815.002 Task 0B.03 Date 5/1/2009	Drawn D. Levulett Analyzed D. Levulett Checked PSK	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Seismic Yield Determination West Levee, Reach C Levee Station 139+23	**Figure** 14-5

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach D\West_D_RS_9501_SeismicK.gsz Date: 5/1/2009 Time: 11:08:00 AM Last Edited By: Levulett, Darcie



amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Seismic Yield Determination West Levee, Reach D Levee Station 159+18	**Figure** 14-6
		Task 0B.03	Analyzed D. Levulett			
		Date 5/1/2009	Checked PSK			

Directory: I:\Project\13000\13815.002\12000 CALC EVAL ANALYSES\Geo Studio\Reach D\West_D_RS_9501_SeismicK.gsz Date: 5/1/2009 Time: 11:08:00 AM Last Edited By: Levulett, Darcie



amec	**Scale** 1 inch = 20 feet	Project Number 13815.002	Drawn D. Levulett	**Stevens Creek** **FEMA Levee Recertification** **Moutain View, California**	**Slope Stability Analysis Result** Seismic Yield Determination West Levee, Reach D Levee Station 159+18	**Figure** 14-7
		Task 0B.03	Analyzed D. Levulett			
		Date 5/1/2009	Checked PSK			